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of Engineers ®
Portland District

PROGRESS REPORT

Mount St. Helens Long-Term Sediment Management Plan for Flood Risk Reduction

Sediment Retention Structure on the North Fork Toutle River

Final June 2010

Report Documentation Page

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EXECUTIVE SUMMARY

When the sediment retention structure (SRS) on the North Fork Toutle River became a run-of-river project in 1998, with all rows of outlet works pipes closed and all flow passing the spillway, a significantly larger amount of sediment began passing the structure. Some of this sediment deposits in the Cowlitz River where it increases flood risk. The 1985 *Mount St. Helens, Washington, Decision Document, Toutle, Cowlitz and Columbia Rivers*, prepared by the U.S. Army Corps of Engineers (Corps) and recommended construction of the SRS, identified dredging in the Cowlitz River as a means to maintain flood risk levels once the SRS became a run-of-river project, as well as providing the option for assessing other long-term alternatives.

The conditions in and around the Cowlitz River are different now than from what they were in 1985. Endangered Species Act issues and a lack of readily available dredge disposal sites make dredging the river difficult and expensive. As a result, a long-term sediment management plan for flood risk reduction was initiated to re-evaluate the sediment conditions and sediment management alternatives.

The Water Resources Development Act of 2000 authorized the Corps to maintain the 1985 Decision Document levels of flood protection for Castle Rock, Lexington, Longview, and Kelso on the Cowlitz River through the year 2035. Shown below are the level of protection (LOP) values authorized by Congress, LOP values in 1996 prior to the SRS becoming run-of-river, and current LOP values. The current LOP values were impacted by both sedimentation in the Cowlitz River and an updated evaluation of hydrology.

Levee Location	Authorized Level of Protection (LOP in years)	1996 LOP Prior to Run-of-River SRS (years)	2009 LOP (years)
Castle Rock	118	212	109
Lexington	167	303	202
Longview	167	370	> 500
Kelso	143	263	470

Interim measures have been implemented to reduce flood risk on the Cowlitz River while the long-term plan is developed. The mouth area of the Cowlitz River was dredged in 2007, 2008, and 2009. The Castle Rock levee upstream of the Arkansas Valley Road Bridge was improved in 2009 by installing a seepage cutoff wall. Coordination with diking districts has increased by adding Cowlitz County to the Corps Portland District Emergency Management list of specified Emergency Operation Centers. This addition will ensure that the Portland District has a liaison dedicated to Cowlitz County for assistance during flood events.

Sediment depositing in the lower Cowlitz River is the problem. The sediment budget for the watershed from the debris avalanche on Mount St. Helens to the mouth of the Cowlitz River was updated in 2009. For water years 2000 to 2007, the average volume of erosion from the debris avalanche was 6 million cubic yards (mcy) per year, the average volume of sediment depositing behind the SRS was 2 mcy/year, and the average volume of sediment passing the SRS was 4 mcy/year. This corresponds to a trapping efficiency of 31%. The trapping efficiency was 92% prior to 1998 when all flow passed through the SRS outlet works. During water years 2000 to 2007, the average sediment load to the Cowlitz River from the Toutle River was 5 mcy/year. This value includes the 4 mcy/year from the SRS, sediment from the South Fork Toutle River and Green River, and sediment from bank erosion. Of that 5 mcy/year entering the lower Cowlitz River, approximately 0.25 mcy/year is deposited in the Cowlitz River and 4.75 mcy/year passed into the Columbia River.

It is important to note that these are average values. The volume of sediment deposited in the Cowlitz River between 2006 and 2008 accounted for 60% of all the sediment deposited over the 2000 to 2007 water years.

In May 2009, the Corps convened a group of six sediment transport and geomorphology experts (the Sediment Evaluation Team) to provide input on the sediment budget and the future sediment yield from the debris avalanche. The Sediment Evaluation Team expressed that it would be reasonable to predict that sediment loading from the debris avalanche will persist at levels between 5 and 10 mcy/year beyond year 2035. They recommended that an analysis be performed to re-estimate the debris avalanche sediment yield decay rate. The Corps has begun such a study, the results of which are expected near the end of 2010.

As stated above, an alternatives analysis has been initiated to determine the most appropriate long-term plan for managing the sediment from Mount St. Helens. Scoping identified the 16 measures listed below as potential measures for evaluation.

1. Debris avalanche stabilization;
2. Elk Rock sediment dam;
3. Sediment plain grade building structures;
4. Sediment plain sump;
5. Raised SRS dam and spillway;
6. Raised SRS spillway;
7. Stabilization of banks;
8. LT-1 sump;
9. Expand floodplain on Toutle River;
10. Modified operation of Mossyrock Dam;
11. Levee improvements;
12. Cowlitz River dredging;
13. Expand floodplain on Cowlitz River;
14. Horseshoe Bend sump or cutoff;
15. Reconnect old channel near mouth of Cowlitz River; and
16. Dikes at mouth of Cowlitz River.

Using two rounds of screening, each measure was evaluated as to the degree to which the measure:

- Reduces flood risk on the Cowlitz River;
- Is low-cost based on considerations of preliminary cost estimates;
- Minimizes impacts to the environment;
- Is reliable;
- Is adaptable to changing conditions; and
- Is acceptable to the public.

After the two rounds of screening, six measures were found to be promising: sediment plain grade building structures, raised SRS dam and spillway, LT-1 bank stabilization, modified operation of Mossyrock Dam, Cowlitz River dredging, and dikes at mouth of Cowlitz River. Of these six measures, two were considered primary measures in that they have the potential to be employed as stand-alone measures. These primary measures included raised SRS and Cowlitz River dredging. Secondary measures may be used to enhance the performance of the primary measures. Grade

building structures, LT-1 bank stabilization, modified operation of Mossyrock Dam (sediment transport flows), pile dikes, and Cowlitz dredging were considered secondary measures.

Analyses done to date show a wide range in estimated costs that reflect the preliminary nature of the conceptual designs and the remaining uncertainties in debris avalanche erosion and sediment transport and deposition into the Cowlitz River. Costs could be up to several hundred million dollars.

The next steps in the alternatives analysis will be to refine and optimize the design of the six remaining measures, advance the modeling of the performance of the measures, and then compare the following alternatives:

Alternative	Primary Measures	Secondary Measures
0	None	Reactive measures
1a	Raised SRS	None
1b	Raised SRS	Short-term Cowlitz dredging
1c	Raised SRS	LT-1 bank stabilization
1d	Raised SRS	Both short-term dredging and LT-1 bank stabilization
2a	Cowlitz Dredging	None
2b	Cowlitz Dredging	Grade building structures
2c	Cowlitz Dredging	LT-1 bank stabilization
2d	Cowlitz Dredging	Flushing flows
2e	Cowlitz Dredging	Pile dikes
2f	Cowlitz Dredging	Some combination

As the alternatives analysis progresses, additional combinations of measures may be developed, as necessary, including redefining measures as primary or secondary and re-evaluating the usefulness of previously screened measures.

The main criteria that will be used to select the preferred alternative include:

- Flood Risk. The alternative must demonstrate a reasonable assurance of maintaining the congressionally authorized levels of protection and not increasing flood risk elsewhere.
- Cost. A least-cost analysis will be performed for the alternatives.
- Environmental Impact. The impact of each alternative on the environment will be considered in the decision-making process.

At the conclusion of the alternatives analysis, a recommendation will be made for the long-term plan for managing the sediment from Mount St. Helens.

COMPLETION OF AGENCY TECHNICAL REVIEW

The District has completed the Mount St. Helens Long-Term Sediment Management Plan for Flood Risk Reduction 2010 Progress Report. Notice is hereby given that an agency technical review, that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the Quality Control Plan. During the agency technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions; methods, procedures, and material used in analyses; alternatives evaluated; the appropriateness of data used and level obtained; and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The agency technical review was accomplished by an independent team from Seattle District. All comments resulting from ATR have been resolved.

Technical Review Team Leader — 6-14-10 Date

Project Manager — 6-14-10 Date

CERTIFICATION OF AGENCY TECHNICAL REVIEW

Significant concerns and the explanation of the resolution are as follows: Concerns were raised about the influence of hydrology and geomorphology on sediment yields, and the uncertainty in the performance of several sediment control measures. NWP has indicated that those items are still under investigation and will be addressed in later reports.

As noted above, all concerns resulting from agency technical review of the project have been fully resolved.

Donald R. Chambers

Chief, Engineering and Construction Division — 6.15.10 Date

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ACRONYMS AND ABBREVIATIONS

AEP	annual exceedance probability
cfs	cubic feet per second
cy	cubic yard(s)
Corps	U.S. Army Corps of Engineers
DWS	design water surface
ESA	Endangered Species Act
FCF	fish collection facility
GBS	grade building structure(s)
HEC	Hydrologic Engineering Center
LiDAR	light detection and ranging
LOP	level of protection
mcy	million cubic yard(s)
mm	millimeter(s)
MSRS	multiple sediment retention structures
NAVD88	North American Vertical Datum of 1988
NGVD	National Geodetic Vertical Datum
NEPA	National Environmental Policy Act
O&M	operation and maintenance
OBM	operating basis mudflow
PDT	Product Delivery Team
PMF	probable maximum flood
PSP	permanent safe (flood) protection
RCC	roller-compacted concrete
RM	river mile(s)
SET	Sediment Evaluation Team
SIAM	System Impact Assessment Model
SRS	sediment retention structure
SWL	safe water level(s)
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey

English to Metric Conversion Factors

To Convert From	To	Multiply by
feet (ft)	meters	0.3048
miles	kilometers (km)	1.6093
acres	hectares (ha)	0.4047
acres	square meters (m^2)	4047
square miles (mi^2)	square kilometers (km^2)	2.590
cubic feet (ft^3)	cubic meters (m^3)	0.02832
feet/mile	meters/kilometer (m/km)	0.1894
cubic feet/second (cfs or ft^3/s)	cubic meters/second (m^3/s)	0.02832
degrees Fahrenheit (°F)	degrees Celsius (°C)	(°F - 32) x (5/9)

1. PURPOSE AND SCOPE

The purpose of the Mount St. Helens Long-term Sediment Management Plan is to develop a plan for managing sediment from Mount St. Helens through 2035, based on considerations of congressionally authorized levels of protection on the Cowlitz River, cost-effectiveness, and environmental impacts. The existing sediment retention structure (SRS) has been operating as run-of-river since 1998 and is now less efficient at trapping sediment. The *Mount St. Helens, Washington, Decision Document, Toutle, Cowlitz and Columbia Rivers* (Corps 1985) identified dredging in the Cowlitz River as a means to maintain flood risk levels once the SRS became a run-of-river project, and also provided the option for assessing other long-term alternatives. The conditions in and around the Cowlitz River are different now than from what they were in 1985. Endangered Species Act (ESA) issues and a lack of readily available dredge disposal sites make dredging the river difficult and expensive. Consequently, a new alternatives analysis was initiated in 2008 to find the best long-term plan for managing sediment from Mount St. Helens. The scope of the work included:

- An update of the sediment budget from Mount St. Helens to the mouth of the Cowlitz.
- A new study to evaluate the future sediment yield and decay rate from the debris avalanche source on Mount St. Helens.
- An evaluation of the current conditions of the Mount St. Helens project features.
- An alternatives analysis including:
    - Development of a list of measures to evaluate;
    - Analysis and screening of measures;
    - Grouping select measures into alternatives; and
    - An alternatives analysis considering flood risk reduction effectiveness, cost-effectiveness, and environmental impacts.
- The National Environmental Policy Act (NEPA) process.

This Progress Report describes the work done to date. In summary, the sediment budget has been updated, the current conditions of the Mount St. Helens project features have been evaluated, and the alternatives analysis has advanced to the grouping of select measures into alternatives. The debris avalanche sediment yield study will be complete by the end of 2010. The alternatives identified in this Progress Report will continue to be analyzed in 2010.

As the Mount St. Helens project is an open construction project, a traditional feasibility study is not planned. Project benefits will not be re-evaluated. A least-cost, environmentally acceptable analysis will be completed to identify the recommended plan. The final alternatives analysis report will be the Decision Document. Appropriate reviews will be completed throughout the planning process. Upon approval of the Decision Document, Design Documentation Reports will be prepared for the recommended long-term plan measures.

2. PROJECT AUTHORIZATION

Under authority of Public Law 99, the U.S. Army Corps of Engineers (Corps) immediately responded to the Mount St. Helens disaster with dredging of the rivers and emergency levee improvements. Congress also authorized interim protection measures in 1983 (Public Law 98-63) for the Corps to maintain at least 100-year protection along the Cowlitz River until an overall solution was in place. These interim measures included construction of temporary debris or check dam type structures

across the North Fork Toutle River (N-1) and South Fork Toutle River (S-1) to immediately reduce the volume of sediment delivered to the Cowlitz, levees were raised along the lower Cowlitz River to prevent flooding, and the Columbia River was dredged to eliminate the threat to navigation. Long-term sediment control facilities were constructed under Supplemental Appropriations Act of August 15, 1985 (Public Law 99-88). The Corps was authorized to construct and operate a SRS near the confluence of the Toutle and Green rivers.

The Corps was directed by Congress to maintain an authorized level of protection (LOP) for four communities along the Cowlitz River that is not less than described in the 1985 Decision Document. These levees are the Castle Rock levee [river miles (RM) 16.10 to 17.55], Lexington levee (RM 6.95 to 9.60), Kelso levee (RM 2.6 to 6.8), and Longview levee (RM 3.1 to 5.5). The authorized LOPs are shown in Table 1. The Water Resources Development Act of 2000 authorized the Corps to maintain these LOPs through the end of the Mount St. Helens project planning period, which is 2035.

Table 1. Authorized Levels of Protection, Cowlitz River Levees

Levee Location	Levee Length (miles)	Percent Chance Exceedance Flood	Average Annual Recurrence Interval (years)
Kelso	5.7	0.70	143
Longview	2.4	0.60	167
Lexington	2.7	0.60	167
Castle Rock	1.5	0.85	118

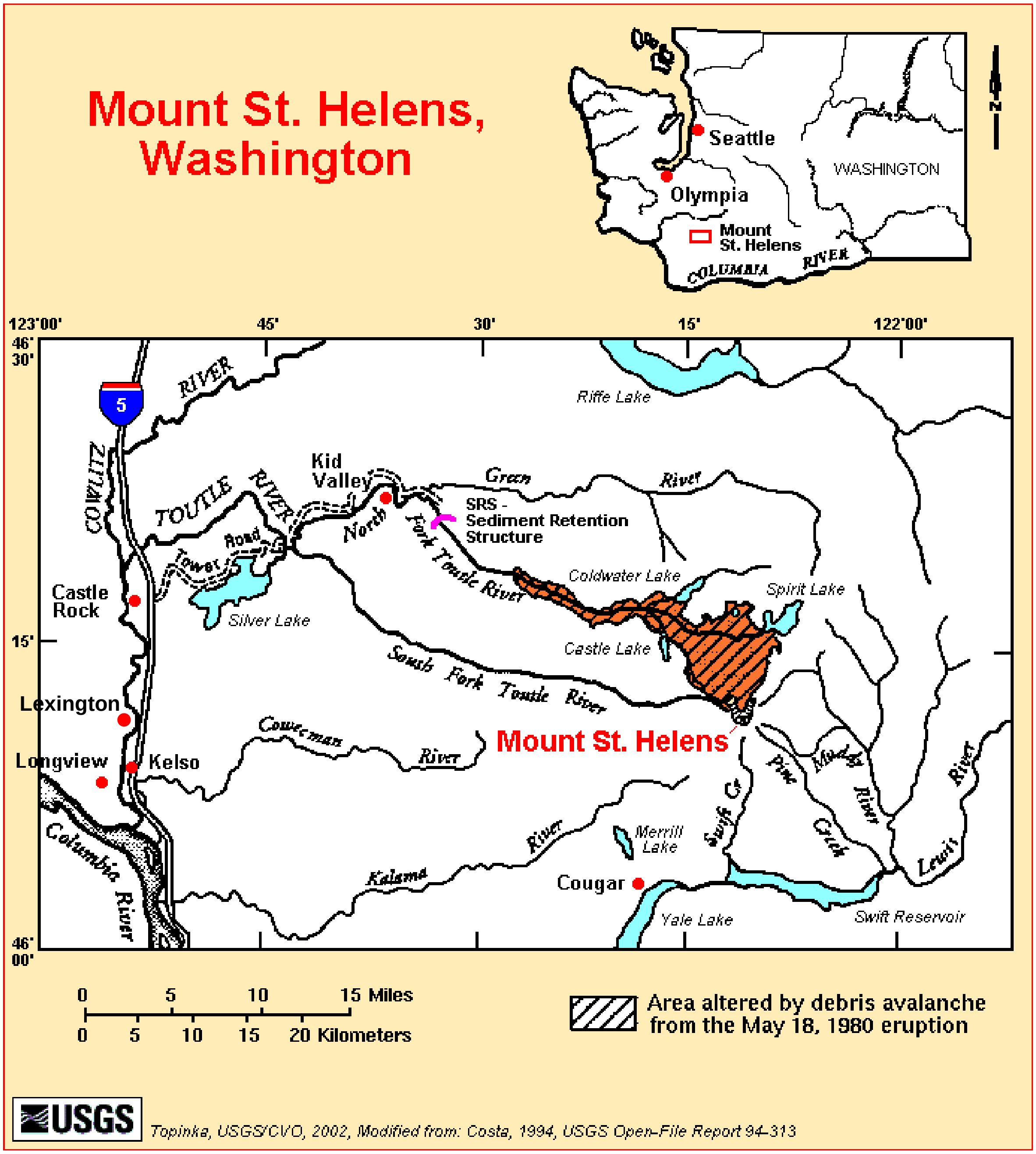
In addition, the Committee on Transportation and Infrastructure of the United States House of Representatives adopted the following Resolution on September 24, 2008 that authorized the Corps to investigate modifications to flood damage reduction for the Coweeman River and levee:

> Resolved by the Committee on Transportation and Infrastructure of the United States House of Representatives, That the Secretary of the Army review reports for Mt. St. Helens including: Lower Cowlitz and Coweeman River Level of Protection Analysis, including Hydrologic Analysis (unpublished analysis/model USACE, Portland District) November 2006, Mount St Helens Engineering Reanalysis, Hydrologic, Hydraulics, Sedimentation & Risk Analysis, Design Document Report April 2002, Mount St. Helens, Washington Decision Document, Toutle, Cowlitz & Columbia Rivers, Oct. 1985, and House Document 2577, Supplemental Appropriations for fiscal year 1985, 99th Congress, and other pertinent reports, to determine whether any modifications of the recommendations contained therein are advisable at the present time in the interest of flood damage reduction for Kelso, Washington.

3. PROJECT LOCATION

The study area encompasses 1,200 square miles in southwest Washington, reaching north from the Columbia River to the headwaters of the Toutle River at Mount St. Helens. A map of the study area is shown in Figure 1. The Columbia River flows east to west through a broad trough between the Cascade and Coast Range mountain ranges. It provides the navigation channel for vessels enroute from the Pacific Ocean to the Ports of Vancouver, Longview, and Kalama Washington, and Portland, Oregon. The reach of primary interest lies from Columbia RM 60 to 72. Lands along both shores, Oregon on the south and Washington on the north, consist of a narrow valley bottom adjacent to low hills. Several small, low-lying islands are located in this reach of the river.

Figure 1. Mount St. Helens and Vicinity



The Cowlitz River and its principal tributary, the Toutle River, are typical of rivers draining the west slopes of the Cascade Range. The terrain is mountainous and, except for clearcuts and areas devastated by the 1980 eruption, heavily forested. The Cowlitz River drains an area of 2,480 square miles including the Toutle River drainage area. Below its confluence with the Toutle, the lower 20 miles of the Cowlitz passes by the cities of Castle Rock, Lexington, Kelso, and Longview, Washington, before entering the Columbia River at RM 67.8.

The Toutle River Basin primarily drains the northwest and southwest slopes of Mount St. Helens and has a total drainage area of 512 square miles at its confluence with the Cowlitz River. The major tributaries of the Toutle River drain 432 square miles. The South Fork Toutle River drains 129 square miles and the North Fork Toutle River drains 303 square miles, including 131 square miles from the Green River. In addition, the lower Toutle River drains 80 square miles. The North and South Fork Toutle rivers have their headwaters on the slopes of Mount St. Helens and carry runoff and sediment westward to the Cowlitz River. The North Fork Toutle River Basin includes three major lakes: Castle, Coldwater, and Spirit (see Figure 1).

The area affected by potential flooding varies from bottomland along the Cowlitz River to uplands at the base of the Cascade Mountains. Industrial riverfront and urbanized property lie adjacent to both the Columbia River and the downstream reaches of the Cowlitz River. Further up the Cowlitz River, adjacent property is less populated, changing from urban to agricultural land use. The upper portion of the Toutle River Basin, except the volcanic and mudflow areas, is managed forestland.

4. HISTORY

4.1. OVERVIEW

The May 18, 1980 eruption of Mount St. Helens dramatically altered the hydraulic and hydrologic regimes of the Cowlitz and Toutle River valleys. Ash fall and the lateral blast from the eruption produced immediate and long-term effects on the hydrology of the Toutle watershed by changing its land cover and runoff characteristics. The excessive amount of sediment produced by the eruption and its aftermath was deposited downstream in the lower Toutle, Cowlitz, and Columbia rivers. The rapid influx of sediment reduced the channel capacities of the rivers affected. This left the communities of Castle Rock, Lexington, Kelso, and Longview in Washington with the potential for major flooding even with normal runoff.

Emergency measures were implemented by the Corps under authority of Public Law 99-88 (August 15, 1985) and interim flood control measures were implemented under authority of Public Law 98-63 (July 30, 1983). Temporary debris or check dam type structures were constructed across the North Fork Toutle River (N-1) and South Fork Toutle River (S-1) to immediately reduce the volume of sediment delivered to the Cowlitz River. Levees were raised along the lower Cowlitz River to prevent flooding, and the Columbia River was dredged to eliminate the threat to navigation.

A Comprehensive Plan (Corps 1983) contained the first in-depth analysis by the Corps of the flooding and sedimentation problems resulting from the eruption of Mount St. Helens. A sediment budget and a deposition analysis were developed as a base for quantifying the size and duration of potential flooding and navigation blockage. A total of 1 billion cubic yards (cy) was estimated to erode in the 50-year study period. From an initial 13 potential measures, some of which were expansions of those used during emergency operations, the following five alternatives were proposed to permanently solve the sedimentation problem:

1. Limited permanent evacuation.
2. Sediment stabilization basins.
3. Multiple sediment retention structures (MSRS) with dredging.
4. MSRS without dredging.
5. Single SRS.

A least-cost analysis based on a 100-year benefits level was performed on five alternatives identified in the 1983 Comprehensive Plan for solving the sediment problem. A single SRS on the North Fork Toutle River upstream from the Green River was the most cost-efficient on the basis of the then predicted erosion rates and timing, and was selected as the most cost-efficient plan to achieve 100-year protection. A subsequent sensitivity analysis confirmed that the SRS remained the most cost-effective option, if the sediment budget was greater than approximately 54% of the predicted amount. This finding, as part of the Comprehensive Plan, was transmitted to the President in October 1983.

In a Memorandum to the Secretary of the Army, dated November 3, 1983, the Assistant Secretary of the Army for Civil Works requested that further analysis concentrate on one or more SRS structures at the lowest feasible site in the Toutle River Basin. It was further directed that other stages or structures should be planned for construction, if and when needed. The rationale for proceeding with the feasibility stage of planning was founded in the unique nature of the problem created by the eruption. Consequently, the uncertainty of predicting erosion rates with field data from a very short post-eruption period necessitated a series of assumptions to predict the sediment budget. The Assistant Secretary stated that notwithstanding the Corps' best estimates of erosion rates, the actual stabilization of the basin by natural processes might occur more rapidly than anticipated. Thus, any programmed solution should provide flexibility to adjust to actual conditions.

Although the SRS was cost-effective over a wide range of the sediment budget, this did not constitute flexibility, as it required a large initial cost. If the movement of sediment was less or slower than predicted, a smaller second state would allow for significant savings of funds required from federal, state, and local treasuries.

A feasibility study was initiated to recommend a permanent solution to the sedimentation and flooding problems for congressional authorization. The sediment budget was revised to indicate erosion of 650 million cubic yards (mcy) of material from the debris avalanche during the 50-year economic project life. A sensitivity analysis again concluded that the SRS was the best plan for handling erosion from the debris avalanche above 65% of the estimated sediment budget.

After reviewing the Feasibility Report (Corps 1984) the Acting Assistant Secretary of the Army concluded that the concerns expressed in the November 3, 1983 Memorandum were still valid. As a result, three options – SRS, staged SRS, and dredging – were to be evaluated during continuing planning and engineering.

Provided below is the *Syllabus* from the 1985 Decision Document (Corps 1985):

> This report analyzes management strategies for dealing with Mount St. Helens related sedimentation and resulting flooding in the Toutle/Cowlitz/Columbia river system. Measures considered include a single sediment retention structure constructed in one stage (SRS) or multiple stages (MSRS), dredging, and levee raises at lower Cowlitz River Valley communities.
>
> The recommended plan is a combination of a SRS (125-foot spillway) at the Green River site on the North Fork Toutle River, minimal levee improvements at Kelso, Washington, and dredging downstream from the SRS during its construction and in later years of the project, when the reservoir has filled and sediment begins to pass over the spillway.

This is the National Economic Development plan, representing the program which will produce the greatest net economic benefits among those considered. In general, its social and physical environmental effects are considerably lower than any management strategy which depends principally on dredging. While requiring mitigation for fish runs into the upper North Fork Toutle River, this plan improves water quality and reduces environmental impacts everywhere downstream from its location. Because much of the sediment will be retained behind the structure, this program will avoid substantial downstream disposal site mitigation.

Of those sites feasible, the Green River site provides the best geologic and farthest upstream location for the earth embankment structure and sediment impoundment area. The structure alone will provide sufficient sediment storage to achieve 167-, 143-, 167-, and 118-year permanent safe flood protection (PSP) at Longview, Kelso, Lexington, and Castle Rock, respectively, over the 50-year project life. The PSP becomes 167-, 143-, 167-, and 118-years at the four communities with levee improvements. The SRS also provides storage for the sediment from a 100-year frequency storm. If monitoring programs suggest more capacity is needed in the reservoir for either rare events (floods for mudflows) or unexpectedly high erosion from the avalanche, it is possible, at additional cost, to raise the spillway and/or crest of the structure when needed.

This program will cost $231.1 million in 1985 dollars. Construction of the SRS, fish bypass, and levees accounts for $65.7 million of those costs. Initial dredging accounts for another $25.4 million and real estate and relocations are $18 million. Other costs, including O&M, monitoring, and outyear dredging total $122 million.

The SRS/levee improvement/dredging strategy recommended is the best alternative when economic, environmental, and engineering considerations are weighed. Preliminary analysis indicates that future raises of the SRS spillway are slightly more economical than outyear dredging along the Cowlitz River. This recommended plan provides more flexibility and safety in managing the unique sedimentation and flooding problem presented by the Mount St. Helens eruption than a dredging only or dredging and minimal levee raise strategy.

4.2. Implementation of the Plan in the 1985 Decision Document

The elements of the Mount St. Helens project are described below.

a. Spirit Lake Outlet Tunnel. Spirit Lake is located about 5 miles north of Mount St. Helens (see Figure 1). By 1982, water in Spirit Lake was rising dangerously high behind a debris dam left by the eruption. A sudden break in the debris dam could have caused severe downstream flooding. The Corps used pumping to relieve water pressure on the debris dam until a permanent solution could be implemented. The permanent solution was a 8,460-foot tunnel to carry water through Harry's Ridge into South Coldwater Creek to maintain a safe water elevation in Spirit Lake. Features of the permanent outlet included the tunnel, a vertical shaft, a gated intake structure, and an approach channel at the intake end. The tunnel was placed in operation in May 1985.

b. Sediment Retention Structure. A SRS was constructed at North Fork Toutle RM 13.2, just upstream of the confluence with the Green River. The SRS features include a dam embankment, outlet works, and spillway. The dam embankment at a crest elevation 1,000 feet National Geodetic Vertical Datum (NGVD) rises 125 feet above the streambed and is 1,800 feet long. The outlet works include approach channel, outlet pipes, outlet works concrete monolith, plunge pool,

and exit channel. The outlet works monolith abuts the right side of the dam embankment. The outlet pipes are 3 feet in diameter and run through the outlet works concrete monolith in six rows of five pipes. The spillway (crest elevation 940 feet NGVD) abuts the right side of the outlet works structure and is 400 feet wide. The original projected SRS sediment storage capacity, when the upstream valley slope reached one-half of the pre-eruption slope, was 258 mcy.

c. Fish Collection Facility. A fish collection facility (FCF) was required as mitigation for blocking upstream fish passage at the SRS. The facility was constructed on the North Fork Toutle River 1.3 miles downstream from the SRS and 0.7 miles upstream from the mouth of the Green River.

d. Levee Improvements. The existing levee at Kelso, which runs from Cowlitz RM 1.3 to 7.0, was raised through improvements to its over-steepened backslopes. The improvements brought the levee up to Corps' standards and provided a nominal 143-year level of protection.

e. Base–Plus Dredging. "Base" refers to the base-level condition that corresponds to the nominal protection levels available in November/December 1983 along the four levees on the lower Cowlitz River. These levees are the Castle Rock levee (left bank from RM 16.1 to 17.55), Lexington levee (right bank from RM 6.95 to 9.6), Kelso levee (left bank from RM 2.6 to 6.8), and Longview levee (right bank from RM 3.1 to 5.5). Base-plus dredging was authorized in both the Toutle and Cowlitz rivers through the year 2035. This broad authorization was intended to encompass emergency measures. No base-plus dredging has been performed on the Toutle River. The last base-plus dredging on the Cowlitz River was in November 1989.

f. McCorkle Creek Pump Station Addition. McCorkle Creek enters the Cowlitz River at Lexington (Cowlitz RM 9.2) via a pumping facility. The eruption and emergency levee modifications impacted the capacity of the McCorkle Creek pumping facility in two ways. First, sediment and debris blocked the gravity flow outlet and raised the base level of the river. Second, the increased levee height resulted in additional head losses. Additional pumping capacity for the pump station was authorized to mitigate flooding along McCorkle Creek.

4.3. STUDIES SINCE IMPLEMENTATION OF THE DECISION DOCUMENT

U.S. Army Corps of Engineers, 1987. Mount St. Helens Sediment Control, Cowlitz, and Toutle Rivers, Washington. Design Memorandum No. 10, Sediment Retention Structure Fish Collection Facility. Portland District, Portland, OR. This design memorandum presented the description, criteria, and design of the fish collection facility constructed by the Corps as mitigation for the SRS. It also discussed interim fish collection.

U.S. Army Corps of Engineers, September 1997. Cowlitz River Flood Hazard Study, Cowlitz County, Washington. Portland District, Portland OR. This study provided estimates of safe protection at authorized communities along the lower 20 miles of the Cowlitz River following the February 1996 flood event. Castle Rock levee was below authorized levels. Kelso, Longview and Lexington levees were above authorized levels. Flood frequency relationships were restudied using a longer period of record.

U.S. Army Corps of Engineers, April 2002. Mount St. Helens Engineering Reanalysis, Hydrologic, Hydraulics, Sedimentation, and Risk Analysis Design Documentation Report. Portland District, Portland, OR. This report reassessed the level of flood protection and determined the risk of flooding was high before the year 2035 at the lower Cowlitz River damage reaches. The study showed when

the LOP at the Castle Rock, Lexington, Longview, and Kelso levees would drop below the authorized levels of flood protection. The report recalculated the 1996 LOP using new index points developed by the Corps. The hydraulic model used for the sediment and LOP analysis was a simplified version of the model developed in the 1997 report. It was noted that the water surface profile for the simplified model compared well to the original model for 1996. The LOP for all index points were greater than the 500-year level. In addition, basic physical and hydraulic data was developed to allow for further alternative analysis.

U.S. Army Corps of Engineers, December 2005. Cowlitz River Basin Hydrologic Summary, Water Years 2003-2004. Portland District, Portland, OR. This report summarized annual rainfall events and the largest instantaneous discharges at the Toutle River Tower Road station and at the Cowlitz River Castle Rock station. The report also showed the annual amount of sediment deposited upstream of the SRS and what is passed downstream.

U.S. Army Corps of Engineers, August 2006. Mount St. Helens Project, Cowlitz River Levee Projects—Level of Protection and Sedimentation Update. Portland District, Portland, OR. This report documented that flood protection provided by the levees along the lower Cowlitz River had been degraded by current sedimentation processes. The observed trend of continued loss of channel capacity was expected to continue and spread upstream, further reducing protection levels. The analysis reports LOP values for 1996, 2003 and August 2006. The 1996 values vary from both the 1997 and 2002 reports. No explanation is provided in the 2006 report; however, discussions with Corps' personnel who worked on the report indicates that an error was found in the Hydrologic Engineering Center's Flood Damage Reduction Analysis (HEC-FDA) runs used in the 2002 report. The 1996 values reported in 2006 are reportedly corrected values from the 2002 analysis. The 2003 and August 2006 values utilized fragility curves developed in 2002, hydrology developed in 1997 and new hydraulic models to reflect deposition in the lower Cowlitz.

U.S. Army Corps of Engineers, July 2007. Mount St. Helens Ecosystem Restoration, General Reevaluation Study Reconnaissance Report. Portland District, Portland, OR. The purpose of this study was to determine if there was a federal interest in pursuing ecosystem restoration actions in the Toutle River watershed, while maintaining congressionally authorized levels of flood protection for communities along the lower Cowlitz River. A range of potential ecosystem restoration measures and the associated costs and environmental benefits were identified and compared to existing conditions. From this report, the Corps decided that there was a federal interest in modifying the SRS spillway to allow for volitional upstream fish passage. Although work began on a design, it was put on hold until the long-term sediment management plan is established.

U.S. Army Corps of Engineers, March 2009. Mount St. Helens Project, Cowlitz River Levees Safe Water Level Study. Portland District, Portland, OR. Congress authorized the Corps to study the influence of Mount St. Helens sediment on the Coweeman River. This study updated the safe water levels for the Castle Rock, Lexington, Kelso, and Longview levees on the Cowlitz River, and the Coweeman levee on the Coweeman River. The Coweeman levee protects the east side of Kelso.

U.S. Army Corps of Engineers, February 2010. Mount St. Helens Project Cowlitz River Levee Systems, 2009 Level of Flood Protection Update Summary, Portland District, Portland, OR. This report provides an updated estimate of the level of protection at Kelso, Longview, Lexington and Castle Rock. New levee fragility curves, hydrologic analysis, and hydraulic model were developed. The Castle Rock levee was below authorized levels. The Kelso, Longview and Lexington levees were above authorized levels.

4.4. PERFORMANCE TO DATE OF MOUNT ST. HELENS PROJECT

The flood risk reduction features of the Mount St. Helens project have been successful to date in preventing flooding within the congressionally authorized protected areas. The main change in performance occurred in 1998, when the SRS outlet works were closed and all flow now passes the spillway. In this operating condition, the trapping efficiency of the SRS is much less and more sediment is passing the structure. Section 5 of this report describes the problem produced by more sediment passing the SRS. Section 6 describes the current condition of the project features and the recent monitoring and trend in levels of protection.

5. PROBLEM

The problem for the Mount St. Helens project is how best to maintain the congressionally authorized levels of protection on the Cowlitz River in light of the increased amount of sediment passing the SRS. The problem is a sediment management issue. Section 6 of this report describes the current condition of the Mount St. Helens project features and the recent monitoring and trend in levels of protection. If no action is taken, sediment will continue to deposit in the Cowlitz River and the levels of protection may drop below the authorized levels.

The Mount St. Helens project is an open construction project. No further evaluation of flood risk reduction benefits will be made for the current study. The 1985 Decision Document (Corps 1985) identified dredging in the Cowlitz River as the recommended approach for managing sediment after the SRS became run-of-the-river. Due to several issues including lack of nearby material disposal sites and ESA issues, it is not clear that dredging remains the best solution. The goal of this study is to identify the best solution given the current conditions for managing the sediment to maintain the pre-determined levels of protection.

Section 8 describes the evaluation of several potential measures for sediment management. The goal is to combine measures into alternatives that each result in a reasonable assurance that the levels of protection will be maintained throughout the project lifetime. These alternatives will then be evaluated in terms of least cost, environmental impact, and public acceptability.

The congressionally authorized project lifetime is to year 2035. However, there is concern that the sediment yield from the debris avalanche may still be high beyond 2035. In May 2009, the Corps convened a group of six sediment transport and geomorphology experts called the Sediment Evaluation Team (SET) to provide input on the sediment budget and the future sediment yield from the debris avalanche. The SET comments are provided in Appendix A. The input on the sediment budget is discussed in Section 7. In terms of future sediment yields, provided below is an excerpt from the team's Comment 9:

> Going beyond 2035: Based on current trends in sediment yield, it is reasonable to predict that sediment loadings in the Toutle-Cowlitz system will persist at levels between 5 and 10 mcy per annum beyond 2035. In this case, an analysis should be performed to indicate just how long it may take for sediment yields to decay to pre-eruption levels, or at least to levels that do not require on-going management actions to prevent them from impacting flood damage potential in the lower Cowlitz valley.

A study of the debris avalanche was initiated to estimate the future sediment yield quantity and timing, which is described in Section 7. The study is expected to be completed late in 2010. In the

evaluation of measures in this report, two timeframes are used: (1) to year 2035 (25 years); and (2) to year 2060 (50 years). The 50-year timeframe is evaluated to provide an indication of differences in decision-making that may occur if consideration is given to extending the authorized project lifetime. Currently, plan development is to maintain authorized levels of protection through 2035.

6. CURRENT PROJECT FEATURES AND CONDITION

This section provides a description of the project features involved in this study. It also covers the history of monitoring of the debris avalanche, the sediment plain, and the Cowlitz River, as well as the history of LOP evaluations for the Cowlitz River. The main project features involved in this study are the SRS, the LT-1 sediment stabilization basin, and the Cowlitz River levees.

6.1. SEDIMENT RETENTION STRUCTURE

The SRS is located at RM 13.2 on the North Fork Toutle River, 30.5 miles above the mouth of the Toutle River. The SRS is a single-purpose structure designed to trap sediment eroding off the debris avalanche on Mount St. Helens. The structure consists of an earth and rock fill embankment dam, an outlet works, and an ungated spillway excavated in rock. The SRS was constructed from 1987 to 1989. The spillway crest is at elevation 940 feet NGVD and the top of dam is at elevation 1,000 feet NGVD. The 60 feet of freeboard is required to safely pass the operating basis mudflow. The spillway width at the crest is 400 feet. As sediment accumulated behind the SRS, the rows of outlet works pipes were buried and closed. The top row was closed in 1998. Since then, all flow passes over the spillway.

The last periodic inspection of the SRS for the dam safety program occurred in 2007. The SRS and all its appurtenant features were found to be in safe operational condition and the project was deemed capable of fulfilling its design purpose. The SRS has currently trapped about 105 mcy of sediment.

6.2. LT-1 SEDIMENT STABILIZATION BASIN

The LT-1 sediment stabilization basin is on the Toutle River, 1.5 river miles above the confluence with the Cowlitz River. Eight sediment basins in the Cowlitz river drainage, including LT-1, were operated from December 1980 to May 1981. Approximately 7.5 mcy of sediment was removed from the river course in this initial period. LT-1 was re-opened during the winter of 1982-1983, and an additional 3 mcy was removed from the river. LT-1 was again operated during the winter of 1983-1984 with an estimated 4.5 mcy removed. The majority of the material excavated from LT-1 was stockpiled on the right bank, with the remainder stockpiled on the left bank. The material was continuously excavated from the river bar during the contract periods (usually winter). The contractor developed a system of berms and levees that allowed control over the location of the active channel. The river was diverted back and forth, usually daily, in order to access newly deposited material. The river was carrying a much higher sediment load during this time period than it is today.

The dredge disposal site on the right bank at LT-1 is now being eroded by the Toutle River. By comparing aerial photos, the estimated erosion volume from 1999 to 2006 was 200,000 cy or approximately 28,800 cy per year on average. Cowlitz County owns the dredge disposal sites on each side of the river at the site. The sites are not developed at this time, except for a small number of houses at the south end of the site on the right side of the river.

6.3. LEVEES

The levees on the Cowlitz River protecting Castle Rock, Lexington, Kelso, and Longview were recently evaluated to update their safe water levels (SWL). The Coweeman levee on the east side of Kelso was also evaluated. Below are findings from the 2009 report:

- For the Castle Rock and Lexington levees, the SWL is at or above the 1980 design water surface (DWS).
- For the Longview levee, the SWL is at the 1980 DWS. For several low sections, it is assumed that the diking district can reliably raise the levee temporarily to achieve a SWL equal to the 1980 DWS.
- For the Kelso levee, the SWL is the 1980 DWS except in two locations: (1) upstream of the sheet pile wall near the upstream end of the levee (North Kelso); and (2) the approximately 1,300 feet long section south of Olive Street and parallel to South River Road (South Kelso). The railroad grade is the SWL for a short distance just upstream of the sheet pile wall near the upstream end of the system. For the section south of Olive Street, with South River Road running along the interior toe of the levee, the SWL is approximately 3 feet below the levee top. The reason for the lower SWL here is the removal of dredge spoils adjacent to the riverward side of the levee and the resulting harmful seepage through the levee predicted at elevated river stages.
- For the Coweeman levee, the SWL is the peak stage at the mouth of the Coweeman River in the 1996 flood event, which is above the 1962 DWS.

In 2008, a preliminary update in hydrology resulted in an increased estimate of flow for a given frequency event on the Cowlitz River. For example, the 100-year flow at Castle Rock was estimated at 116,000 cubic feet per second (cfs) as compared to the 97,000 cfs estimated in 1997, an increase of 20%. Given this information and the increase in sedimentation in the Cowlitz River, the Corps decided to raise the SWL for the critical reach of the levee at Castle Rock. The estimated LOP for this reach (the levee north of the bridge) was below 100 years. The SWL for this reach could be raised by constructing a seepage cutoff wall down the middle of the levee into the foundation. A cement-bentonite cutoff wall was constructed in the fall of 2009 in this reach. The SWL in this reach is now the top of the levee. The critical reach for the Castle Rock levee system is now a short length adjacent to a retaining wall downstream of the bridge. Based on the 2009 update, the LOP for this reach is 109 years, slightly below the authorized 118 year level of protection.

6.4. MONITORING

The *Mount St. Helens Sediment Control, Cowlitz and Toutle Rivers, Washington, Sediment Retention Structure Sediment Ranges, Design Memorandum No. 11* (December 1986) establishes a monitoring program to determine sediment deposition upstream and the resulting downstream impacts of the SRS. Downstream impacts include determination if the designed LOP is being maintained along the lower Cowlitz River. The monitoring program also provides the data required for planning and designing of additional remedial actions if needed. Components of the system have evolved with changes in project conditions. Primary monitoring elements include continuous flow and sediment gages on the Toutle and lower Cowlitz River, cross section and terrain data for the SRS, and cross section data for the lower Cowlitz River that are reported in annual hydraulic summary reports.

6.4.1. Monitoring Reservoir Sedimentation

Sediment Ranges and Terrains. Behind the SRS, a depositional zone extends from RM 13.3 and upstream for approximately 7 miles up to the (now defunct) SRS (N-1) built in 1980. To monitor the changes in channel characteristics, the sediment plain has been consistently analyzed using 25 cross sections since the construction of the SRS. In addition to these cross sections, terrain data collected early on by orthophotography and later by LiDAR, have been used to verify the volume estimates made using cross-sections and to investigate 2-D effects. Remote sensing has become price competitive in recent years, such that traditional survey of cross sections has ceased and annual estimates of deposition are made exclusively from terrain products.

Gradation of Sediment Deposits. Sediment sampling of the SRS depositional plain was intended primarily as a means to evaluate and potentially modify operations of the SRS. Samples have been collected in the 10 years since construction of the SRS in 1987. Initial efforts were considerably more robust in scope. The last significant collection effort took place in 1999 after the SRS had begun flowing through the spillway.

Meteorological Stations. The Corps Portland District office operates two weather stations in the Toutle River Basin: one at the SRS and one at Coldwater Ridge in the North Fork Toutle Basin. The SRS station records air/water temperature, precipitation, wind speed/direction, and the water level at the spillway. Coldwater Ridge records air temperature, precipitation, and wind speed/direction.

6.4.2. Monitoring Downstream Impacts

Cowlitz Hydrosurvey. Bathymetric surveys of the lower 10 miles of the Cowlitz River has been periodically performed since the eruption. Nine full reach and four partial reach cross-section data sets have been collected since the end of major recovery dredging in 1989. Bathymetric survey was identified as part of the primary monitoring program to estimate the impacts of sedimentation and sedimentation process on level of flood protection. Datasets collected include:

2009 August
2008 February/March/May
2006 December
2006 April (lower 10 miles)
2003 August
2000 October (limited dataset)
1999 June (limited dataset)
1998 June (limited dataset)
1996 Summer
1992 July
1991 August
1990 May
1989 April

Bed Gradations. Bed gradation data along the lower 20 miles of the Cowlitz River is collected periodically to determine the quality of material in the river. This data can be used to determine if bed armoring is occurring as well as identifying the types of material that are depositing in the lower Cowlitz. Five datasets have been collected in the lower Cowlitz since the SRS began passing water over the spillway.

Bed Gradation Data	# Samples	Extents
1992 August	44	RM 0.0 to RM 19.7
2000 October	05	RM 1.1 to RM 15.5
2004 June-August	08	RM 1.1 to RM 18.8
2005	17	RM 1.7 to RM 19.8
2007 January	10	RM 0.3 to RM 8.5

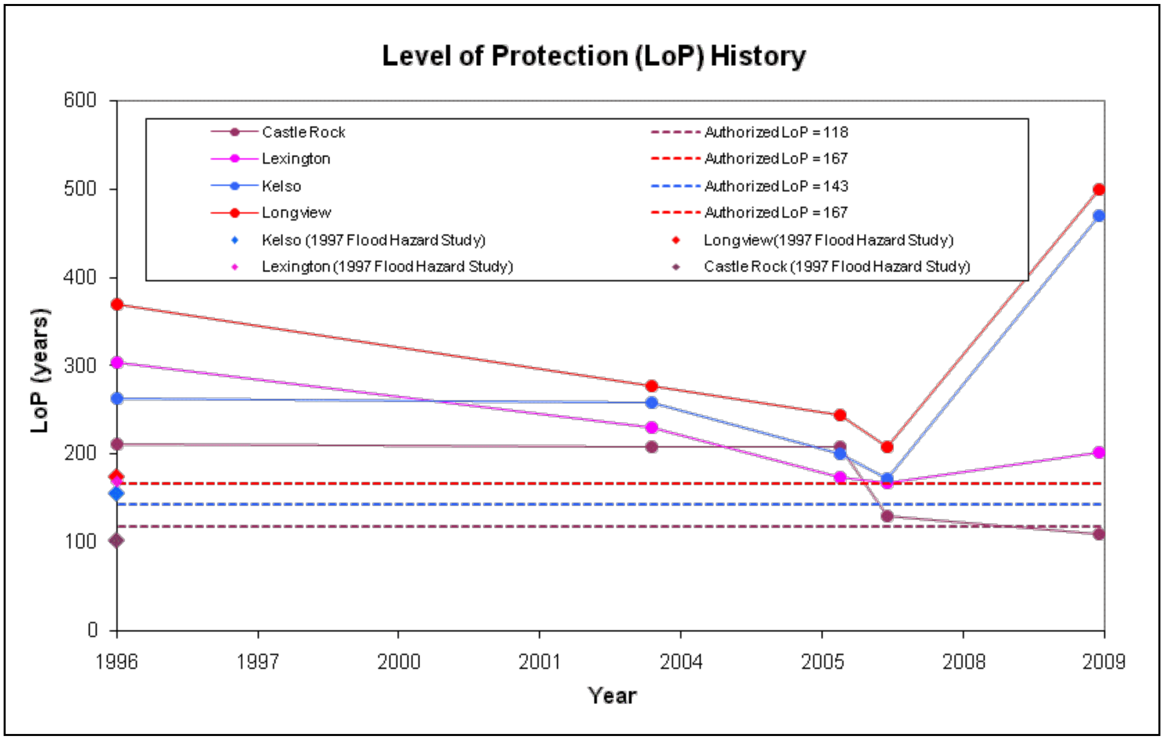
Flow and Sediment Gages. The U.S. Geological Survey (USGS) operates stage, discharge and sediment monitoring stations in the Toutle River Basin. The gages at Toutle River at Tower Road (14242580) and at South Fork Toutle at Toutle, Washington (14242580) have the longest continuous datasets that include sediment discharge. The daily sediment discharge data measured by these gages is an important tool in monitoring function of the SRS. A long-term stage and flow gage at Castle Rock at RM 17 on the lower Cowlitz River provides the primary flow data for the four authorized levees. Several other gages have been added and removed as needs and funding changes. A stage, flow and sediment monitoring station immediately below the SRS was reinstalled in 2007 and now measures flow and sediment outputs from the SRS. A flow and sediment gage was initially identified immediately below the debris avalanche to monitor influx to the SRS. A gage was installed and maintained at Elk Rock for a short period of time before the effort was abandoned due to difficulty and expense in acquiring accurate readings. The rapidly and dramatically changing and very wide section made flow and transport estimates unreliable.

Corps Stage Gages. The Mount St. Helens Engineering Reanalysis (Corps 2002) recommended a system of seven stage gages to monitor sediment impacts to flow levels from the mouth of the Cowlitz River to the confluence of the Cowlitz River with the Toutle River. Five water level loggers were installed on the Cowlitz River during October 2002. These five gages along with USGS gage No. 14243000 (Cowlitz River at Castle Rock) and the National Weather Service gage No. 454131 (Cowlitz River at Kelso) make up the seven-stage gage system (Corps 2005). The system of gages was installed to track potential changes in channel capacity due to sedimentation by specific gage analysis, and serves other purposes including model calibration/verification.

Level of Protection. The Corps was directed by Congress to maintain an authorized LOP in four communities along the Cowlitz River that is not less than described in the 1985 Decision Document (Corps 1985). Congressional direction provides that the Corps must maintain the LOP for the Cowlitz River levee systems through the end of the project planning period, which is 2035. The authorized LOP for the Cowlitz River levees (see Table 1) are expressed as recurrence interval floods that result in the levee system capacity exceedance or failure. Figure 2 shows the LOPs as evaluated at different points in time since 1996.

The procedure and methodology for determination of the LOP for the Cowlitz River levees since 1997 is a risk-based analysis approach at designated index locations along the levee system. Prior to 1997, including the analysis on which the authorization is based, a deterministic approach was used to calculate LOP. The release of ER 1105-2-101 in March 1996 required that all flood damage reduction studies adopt a risk-based analysis. Index locations were chosen based on a detailed assessment of levee conditions and represent critical locations along the levee that provides the least amount of flood protection. Three key factors were involved in the risk based analysis of determining the current LOP for the Cowlitz levee system: geotechnical or levee risk, hydrologic risk, and hydraulic risk. The HEC-FDA incorporates all three components of risk to compute the LOP in terms of a recurrence interval flood for each index point within the levee reach.

Figure 2. Level of Protection History, Cowlitz River Levees (line represents latest data)



The Corps periodically updates the LOP for the Cowlitz River levees as part of the ongoing activities of the Mount St. Helens project. The 2009 update is the most comprehensive update since the 1997 Cowlitz River Flood Hazard Study. Since 1997, however, the LOPs have been updated periodically. For each update, combinations of new data were used to assess current level of protection levels. Table 2 summarizes all the LOP updates that have occurred since 1997 and the new data incorporated for the corresponding update. Table 3 provides a history of regulated peak discharges for the Cowlitz River at Castle Rock.

There are two main reasons the estimated LOP increased for Lexington, Kelso, and Longview between the 2007 evaluation and the 2009 evaluation. The first reason is an improved understanding of the channel's roughness in the lower part of the Cowlitz River during high flow events gained by calibration of the hydraulic model to high water marks observed during the January 2009 flood event, the largest flood event observed since 1996. During high flows, the sand bed in the lower 10 miles of the Cowlitz River changes its bedform regime and becomes smoother, resulting in relatively lower river stages as compared with previous estimates of LOP. This phenomenon is described in the 2009 LOP update report. The second reason is the recent dredging performed in the lower 5.7 miles of the Cowlitz River. Section 11 of this report describes the dredging activities. These two factors did not influence Castle Rock, which is much further up the river. At Castle Rock, the LOP decreased between 2007 and 2009 due to sedimentation in the river and reanalysis of levee fragility.

Table 2. Cowlitz River Level of Flood Protection Updates

Level of Protection (LOP) Update	Hydrographic Survey	Levee Fragility Curve Restudy	Hydrologic Restudy
September 1997 Cowlitz Flood Hazard Study	June 1996 (survey RM 0-20)	1992 CENWP Analysis (except at Longview)	Yes
Cowlitz River Levee Projects, LOP and Sedimentation Update, August 2006	June 1996 (survey RM 0-20)	2002 CENWP Restudy	No
	August 2003 (survey RM 0-20)	2002 CENWP Restudy	No
	April 2006 (survey RM 0-10)	2002 CENWP Restudy	No
2007 LOP Update (unpublished)	December 2006 (survey RM 0-20)	2002 CENWP Restudy	No
2009 LOP Update *(Current Update)*	August 2009 (survey RM 0-20)	2009 *Cowlitz River Levees Safe Water Level Study*	Yes

Table 3. History of Regulated Peak Discharges for Cowlitz River at Castle Rock

Percent Chance Exceedance	Cowlitz River at Castle Rock Peak Flow (cfs)		
	1985 Hydrology Report in Cowlitz Dredging Memo No. 4	1997 Flood Hazard Study	2009 LOP Update Summary
50	60,300	55,000	46,000
10	79,300	74,000	80,000
2	91,500	90,000	108,000
1	102,000	102,000	113,000
0.5	Not reported	120,000	124,000
0.2	151,000	241,000	160,000

7. POLICY CONSIDERATIONS

Several policy issues have been identified in the development of this Progress Report. Following is a summary of key policy issues that must be addressed before developing a long-term implementation plan to manage sediment in the lower Cowlitz.

- Decision-makers should be fully aware of the impact of changes in how LOPs are currently calculated as compared to when the original 1985 Decision Document was developed. In 1985, a deterministic approach was used to calculate LOPs, whereas a risk-based approach to calculate LOPs is now required under current Corps guidelines. This change in evaluation approach and updated hydrology information results in higher water surface elevations along the lower Cowlitz River, even before considering the impacts of sediment deposition. This could result in additional costs to maintain authorized LOP beyond what was originally anticipated in the 1985 Decision Document. Put another way, the 1985 Decision Document plan was formulated to address future sediment deposition in the lower Cowlitz River. The changes in the LOP evaluation process and updated data have significant impacts on water surface elevations and required actions to maintain authorized LOPs. The water surface elevations associated with a given LOP, as identified in the 1985 Decision Document, have

increased resulting in providing additional protection over that addressed in the 1985 Decision Document, potentially increasing the cost to the federal taxpayer.

- The 1985 Decision Document outlined a strategy to manage sediment through year 2035. The existing authorization does not go beyond 2035. Given current analyses, sediment infill in the lower Cowlitz River may be a significant concern beyond 2035. Decision-makers need to be aware of the implications as this study progresses because different periods of analysis often result in different outcomes.
- As the alternatives analysis continues and the least-cost alternative is identified, the cost of this alternative will have to be compared to the authorized project budget limit (Section 902 limit). If the 902 limit is exceeded, a Post Authorization Change report will be required.
- Flood risk to Kelso, Washington, associated with the Coweeman River is somewhat influenced by Mount St Helens sediment from the Cowlitz River. Currently, the Coweeman River is not included as a part of the authorized Mount St. Helens project.
- Use of models to complete analyses and development of decision documents will require appropriate reviews. The specific needs will need to be defined and completed.

8. SEDIMENT BUDGET

The purpose of the Toutle/Cowlitz River Sediment Budget Report (Corps 2009) is to present a sediment accounting that identifies the existing watershed sediment sources, pathways of sediment transport and sinks of temporary storage of sediment. In future studies, this sediment budget will provide a framework for identifying, screening and evaluating potential alternatives. A sediment budget is an accounting of the sediment movement into and out of a selected location.

In the Toutle/Cowlitz Rivers watershed, an accounting of the sediment load has been conducted beginning upstream within the debris avalanche plain along the North Fork of the Toutle River and continuing downstream to the mouth of the Cowlitz River adding estimated sediment loads from various sources along the way. Estimation of sediment sources was the result of careful examination of all available data within the system. Suspended sediment data, sediment samples, bathymetric data along the Cowlitz, light detection and ranging (LiDAR) data and other aerial surveys, and ground survey are included in the information used to formulate appropriate sediment sources. Temporal density of the information is highly variable and in some cases the data is sparse. To develop a sediment budget with available data, judgments have been made of the usefulness of the data and relevance of the time periods over which the data is most valid.

The sediment budget was formulated under the assumption that the North Fork, South Fork, and Toutle Rivers act as a conduit for efficiently moving sediment mainly sands, silts, and clays to the Cowlitz River. Sediment depositing in sink locations along the Toutle during dry hydrologic conditions will likely return to suspension and be delivered to the Cowlitz given time. Simulation of sinks or routing of sediment through the system to the Cowlitz requires a mobile bed sediment transport model, which was not included in the scope of this report.

In addition to LiDAR and gage analyses necessary for the sediment budget, a supplementary investigation of the historical survey data and gradation analyses of the sediment filling the SRS has been included. Although this supplemental topic was not directly utilized in the sediment budget, the perspective offered by the additional data is of significant value to the report.

The sediment budgets were calculated by mass (tons) and by grain size. The sediment budgets appear as tabular spreadsheets with sediment sources and sinks listed along the left column. All values are

determined arithmetically; particle routing considering mass and hydraulic capacity is not included in the sediment budgets. Key results and conclusions of the analyses presented in the Sediment Budget Report (Corps 2009) are summarized below:

- Evidence of decay in the rate of debris avalanche erosion was not found to be significant in available data collected during the past 20 years. Cumulative debris avalanche erosion predicted by 2035 ranges from 125 to 227 mcy, with a mean value of 165 mcy. Calculation of debris avalanche erosion was conducted using surface comparisons that were found to have an uncertainty of ±15%.
- The SRS filled to the spillway crest with sediment in 1998; since then, sediment moving through the spillway was approximately 80% of the total sediment sources contributing to the Toutle/Cowlitz system. Sediment output from the SRS from 1999 through 2007 was estimated to be approximately 46% silts and clays, 40% fine sands, 6% medium sands, and 8% coarse sands.
- The total sediment load delivered to the Cowlitz River at the mouth of the Toutle River from 1999 through 2007 was estimated by the sediment budget to be 56.2 million tons and was composed of 41% silts and clays, 40% fine sands, 9% medium sands, 8% coarse sands, and 2% gravel. Uncertainty associated with the total load ranges from ±17% and ±72%, with an average uncertainty of 28%. Uncertainty in the load by grain size is considerably larger.
- The cumulative sediment load forecast between 2008 and 2035, with uncertainty incorporated, at the mouth of the Toutle River was predicted to be between 79 and 370 million tons. The total 95% limit ranges from 122 to 237 million tons with a mean of 173 million tons.
- The sediment budget methodology provided an efficient, first-approximation method for estimating total sediment yield along a river system. Primary limitations in the method were the temporal density of the data relative to the temporal density of the estimates required, and the inability of the method to include hydraulic sediment routing by grain size.
- Local sediment sinks have been observed in a few locations along the Toutle, North, and South Fork Rivers; however, based upon analysis of stream power, critical shear, suspended sediment data and field observations, these sinks were thought to be relatively small in comparison to the sediment sources.
- Sediment deposition rates in the lower Cowlitz River have increased since 2003. The most recent analysis period, 2006 to 2008, showed the highest depositional rates of all analysis periods. The high depositional rates observed between 2006 and 2008 are likely due to very high sediment loadings associated with the November 2006 storm event and subsequent dredging activities, and likely do not represent a steep rising trend in deposition. While the highest rates were in the lower 2 miles, a high persistent depositional rate was observed in the lower 10 miles and again in the upper 5 miles.
- Sediment deposition occurring in the lower Cowlitz was found to be primarily medium and coarse sands. Discrepancies were found between the quantity of medium to coarse sand sampled by USGS gages and the quantity of those particles found in the sediment at the mouth of the Cowlitz River.
- Approximately 40% of the predicted sediment yield at the mouth of the Toutle River is in the silt and clay range.

9. MEASURES EVALUATION

9.1. Evaluation Process

The measures evaluation process consisted of three primary phases. The first phase involved a review of existing information and a measures brainstorming workshop. The 2-day workshop was held in December 2008. After a day of field trips, participants from the Corps Portland District, the Biedenharn Group (contractor), and Cowlitz County spent a day brainstorming measures that could be implemented to manage sediment and reduce flood risk on the Cowlitz River. From this workshop, 16 measures were selected for evaluation. A first level screening was performed on these 16 measures to evaluate the degree to which each measure:

- Reduced flood risk on the Cowlitz River;
- Was cost-effective;
- Minimized impacts to the environment;
- Was reliable;
- Was adaptable to changing conditions;
- Protected cultural resources; and
- Was acceptable to the public.

The first level screening was done in light of the sediment budget and comments from the SET. During the first level screening, it became clear that 9 of the 16 measures had major shortcomings related to one or several of these factors (typically reliability, cost-effectiveness, and/or ability to significantly reduce flood risks), and should be dropped from further analysis. These measures were set aside from further consideration but may be revisited in the future if conditions change.

For the seven remaining measures, a second level screening was performed where conceptual designs and cost estimates were developed. Limited hydrologic, hydraulic, and sediment transport modeling was performed. Measures were evaluated in terms of the same factors as in the first level screening. At the end of the second level screening, the measures were grouped into alternatives. These alternatives will be analyzed to select the recommended plan.

9.2. Potential Measures

The 16 measures evaluated during the first level screening included:

1. Debris avalanche stabilization;
2. Elk Rock sediment dam;
3. Sediment plain grade building structures;
4. Sediment plain sump;
5. Raised SRS dam and spillway;
6. Raised SRS spillway;
7. Stabilization of banks;
8. LT-1 sump;
9. Expand floodplain on Toutle River;
10. Modified operation of Mossyrock Dam;
11. Levee improvements;
12. Cowlitz River dredging;

13. Expand floodplain on Cowlitz River;
14. Horseshoe Bend sump or cutoff;
15. Reconnect old channel near mouth of Cowlitz River; and
16. Dikes at mouth of Cowlitz River.

The general location of each measure is shown in Figure 3. Measures 1, 2, 4, 6, 7, 9, 11, 14, and 15 are described in Section 8.4. These nine measures were not advanced to the second level screening. Measures 3, 5, 8, 10, 12, 13, and 16 were carried forward for second level screening and are described in Section 8.5.

9.3. Modeling Tools

The sediment budget described in Section 7 (Corps 2009) provides the basic sediment and hydrologic input data for analysis of proposed alternatives. The budget defines the scale and quality of the sediment flux throughout the system as well as basic information on variability. The budget assumes that future sedimentation in the system through the planning period (ending 2035) will be similar to that observed since SRS outflows began running through the spillway. Since the budget is limited to data observed in the existing condition, other modeling or analytic analyses are needed to predict depositional responses to proposed projects. For second level screening, a suite of models being developed for alternative analysis are used to investigate effectiveness of proposed measures. Since the modeling tools are in development and not final at this time, the results should be viewed as trends in lieu of absolute values. The measures being modeled are likewise early in development and reflect a preliminary concept. Three main modeling tools are used to estimate the effectiveness of the proposed measures: spreadsheet calculation, one-dimensional hydraulic and sediment modeling, and two-dimensional hydraulic and sediment modeling.

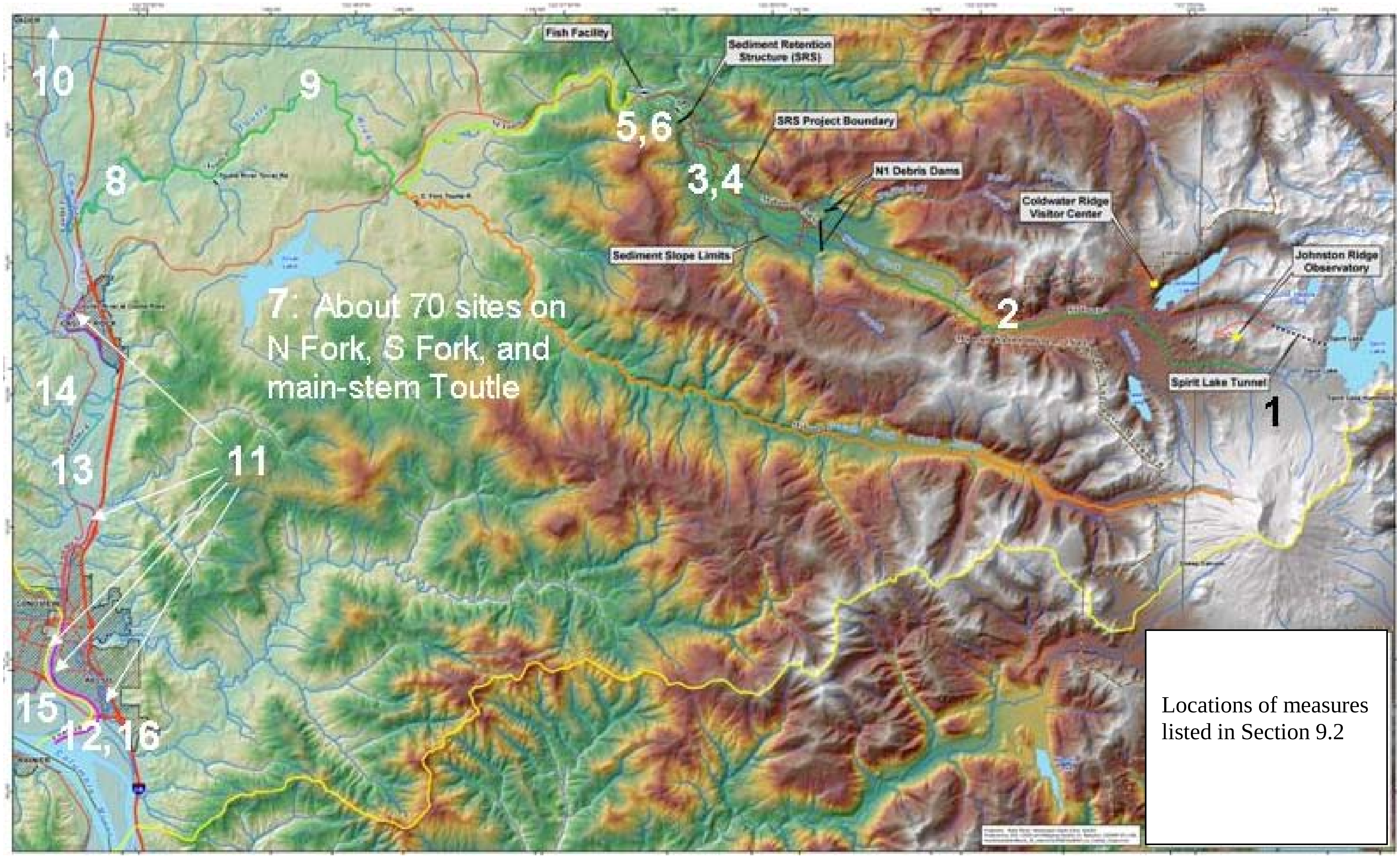
9.3.1. Spreadsheet Calculations

Three measures that trap sediment – raised SRS, valley-wide grade building structures upstream of the SRS, and LT-1 sediment sump – lend themselves well to spreadsheet style calculations for initial screening. A trapping efficiency calculation per grain size is made and used to modify the sediment budget projections through the planning period.

9.3.2. One-dimensional Hydraulic and Sediment Modeling

A mobile bed HEC-RAS model of the lower 20 miles of the Cowlitz River has been created using 2008 bathymetric data and bed gradation data from 2005 and 2007. The model has been hydraulically calibrated to the January 2009 out-of-bank high water event. Sediment loading data from the Toutle River is generated using information contained in the sediment budget. The model allows for investigation of measures being performed in the lower Cowlitz, but also can be used to measure the effects of measures performed upstream. The Cowlitz River mobile bed model is used to analyze modified flow releases from Mossyrock Dam. By modifying the upper Cowlitz input hydrograph, changes in depositional rates compared to the existing condition can be determined.

Figure 3. Measures Location Map



9.3.3. Two-dimensional Hydraulic and Sediment Modeling

A depth averaged two dimensional hydrodynamic and sediment transport model (MIKE21-C from DHI Software) has been created for Cowlitz-Columbia confluence including the lower 4.5 miles of the Cowlitz River, 6 miles of the Columbia River, and Carrols Channel. Two-dimensional modeling is well suited to determine sedimentation processes in the tidally influenced lower portion of the Cowlitz and Columbia rivers. The model is based on relatively high density bathymetric data collected in 2008. The two-dimensional model is used to evaluate the effectiveness if a pile dike program in the lower 5 miles of the Cowlitz River.

9.4. FIRST SCREENING

9.4.1. Measure 1: Debris Avalanche Stabilization

The estimated initial volume of the debris avalanche is 3 billion cy. It covers 32 square miles of area and is 17 miles long, over 600 feet deep in some locations, averages 150 feet deep, tapers down to 10 feet deep at the toe, and has an overall slope of about 3%. The material in the debris avalanche varies in size from clays to boulders. Its gradation is about 40% to 45% coarse sand or larger [i.e., larger than 2 millimeters (mm)], 40% to 45% sand, and 10% to 20% fines (less than 0.062 mm).

The purpose of this measure is to stabilize this massive sediment source in place, to reduce erosion of the sediment into the North Fork Toutle River. Three stabilization techniques were proposed: (1) seeding and planting, (2) soil amendment, and (3) channel bank stabilization. Seeding and planting the debris avalanche was evaluated as a potential measure during development of the comprehensive plan in 1983. The measure involves fertilizing the nutrient-poor sediment and seeding and planting vegetation to stabilize the upper layer of the debris avalanche. Soil amendment involves mixing a binding agent such as cement into the debris avalanche sediment to increase its resistance to erosion. A large portion of the debris avalanche surface area would be treated to a shallow depth. The intent of this approach is similar to that of seeding and planting: to stabilize the upper layer.

The main form of erosion is erosion within the channels extending into the debris avalanche. Channel widening in particular is the dominant mechanism. In 1983, main channel erosion was noted as the dominant form of erosion. The same conclusion was reached by the SET in 2009. The channel banks are high and steep, as shown in Figure 4. Several mechanisms contribute to bank caving associated with channel widening: scour at the toe of the banks, saturation of the banks due to heavy rainfall, and rapid drawdown action as high water recedes from the saturated slopes. One idea that was discussed is attempting to stabilize these banks from further caving. If the debris avalanche could actually be stabilized, there could be a large improvement to flood protection on the Cowlitz River.

The debris avalanche stabilization measure is not considered feasible. Measures that stabilize the upper layer—seeding and planting and soil amendment—would not target the main form of erosion, which is channel erosion, and would be undermined by channel erosion and caving banks. Given the height and erodible nature of the bank soils, and the potential for continued incision, it is not considered feasible to stabilize the channel banks. In addition, the measure is not compatible with the 1982 Mount St. Helens National Volcanic Monument Act (Public Law 97-243), which inhibits actions that would disrupt the natural geological and ecological processes within the Monument.

Figure 4. Channel in the Debris Avalanche

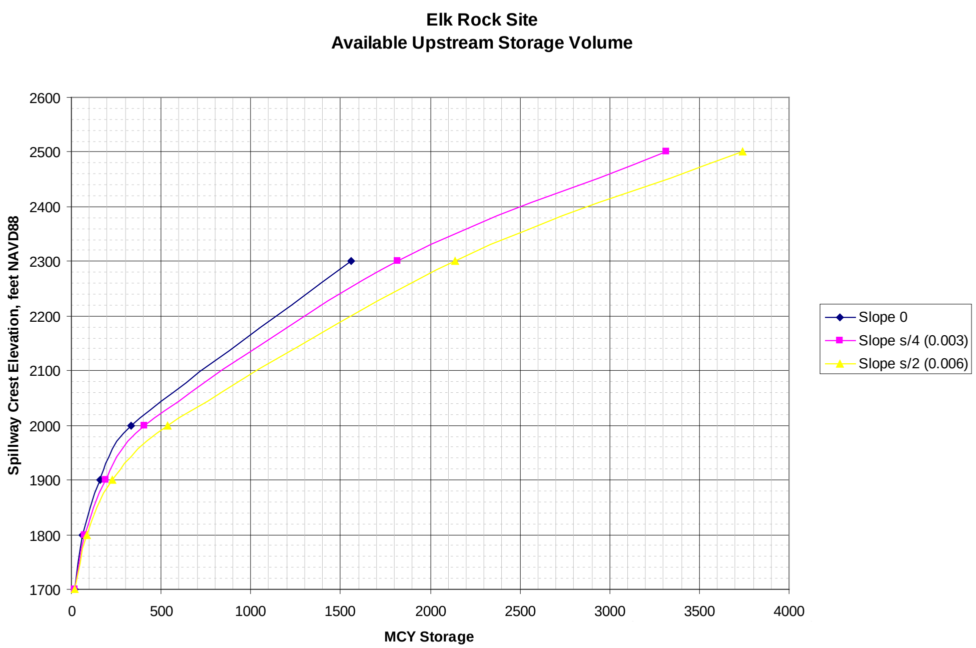
9.4.2. Measure 2: Elk Rock Sediment Dam

Elk Rock is at the approximate location of the toe of the debris avalanche. This measure involves building a sediment dam at the Elk Rock constriction to trap sediment eroding off the debris avalanche. At this location, the valley bottom elevation is approximately 1,590 feet. Figure 5 shows curves for sediment storage volumes as a function of spillway crest elevation for three final sediment slopes behind the structure: 0, 0.003, and 0.006. To trap 500 mcy of sediment up to a final slope of zero, for example, the spillway crest would be built to elevation 2,040 feet.

The sediment dam would include an earth embankment, outlet works, and spillway. The foundation for the earth embankment would be prepared by removing the 1980 mudflow deposit and the overlying sediment. The outlet works would be a concrete structure founded on rock. The spillway would be excavated in rock on one of the abutments. To trap 500 mcy of sediment up to a final slope of zero, the height of the structure from the existing valley bottom to the spillway crest would be approximately 450 feet. The top of the dam would be about 50 feet above the spillway crest to provide freeboard for passing a mudflow. Thus, the overall height of the structure would be about 500 feet. The length of the structure across the valley would be about 3,500 feet.

Figure 5. Spillway Crest/Storage Relationships for Elk Rock Sediment Dam

Storage Volume : Elk Rock



Elk Rock Storage Volumes (Spillway Crest Elevations in NAVD88)

	Sediment Slope		
Spillway Elevation	**Flat (0)**	**s/4 (0.003)**	**s/2 (0.006)**
1700	16	18	20
1800	61	72	86
1900	161	189	229
2000	333	409	538
2300	1560	1816	2138
2500		3317	3738

*Note: Sediment Volumes given in MCY

If built as described above, the Elk Rock Sediment Dam would trap a large volume of sediment close to the debris avalanche source, resulting in a very positive impact on flood protection in the Cowlitz River. Compared to raising the existing SRS, which is a similar measure, the Elk Rock Sediment Dam would have less impact to fish, as raising the SRS would bury significant portions of tributaries with habitat located between the SRS and Elk Rock. However, compared to the cost of raising the SRS to trap the same volume of sediment, the cost of constructing the Elk Rock Sediment Dam would be much greater as it would involve building a new foundation and a much higher structure due to the increased slope of the river closer to the mountain. The higher Elk Rock Sediment Dam would pose a higher risk than a lower raised SRS dam if the dam were to fail due to a mudflow or earthquake. The Elk Rock dam would have to be combined with a measure or measures near or on the Cowlitz River, such as a sump on the Lower Toutle or dredging in the Cowlitz, to handle the existing potentially mobile sediment below Elk Rock, including the sediment plain behind the SRS, until this potentially mobile sediment is flushed through or otherwise removed.

A major reason against construction of the Elk Rock Sediment Dam is that it conflicts with the 1982 Monument Act. Elk Rock is just within the downstream boundary of the Monument. The Monument Act states, “The Secretary shall manage the Monument to protect the geologic, ecologic, and cultural resources, in accordance with the provisions of this Act allowing geologic forces and ecological succession to continue substantially unimpeded” [Section 4(b)(1)]. Building the dam, and the disruption of the natural erosion of sediment from the Monument, would violate the Act. However, the Monument Act further states that, “The Secretary may take action to control…agents that might…cause substantial damage to significant resources adjacent to the Monument [Section 4(b)(2)(B)] and that, “Nothing in this Act shall prohibit the Secretary from undertaking or permitting those measures within the Monument reasonably necessary to ensure public safety and prevent loss of life and property” [Section 4(b)(3)]. The Elk Rock Sediment Dam would likely have to be far superior to any other option in order for the Secretary to permit its construction.

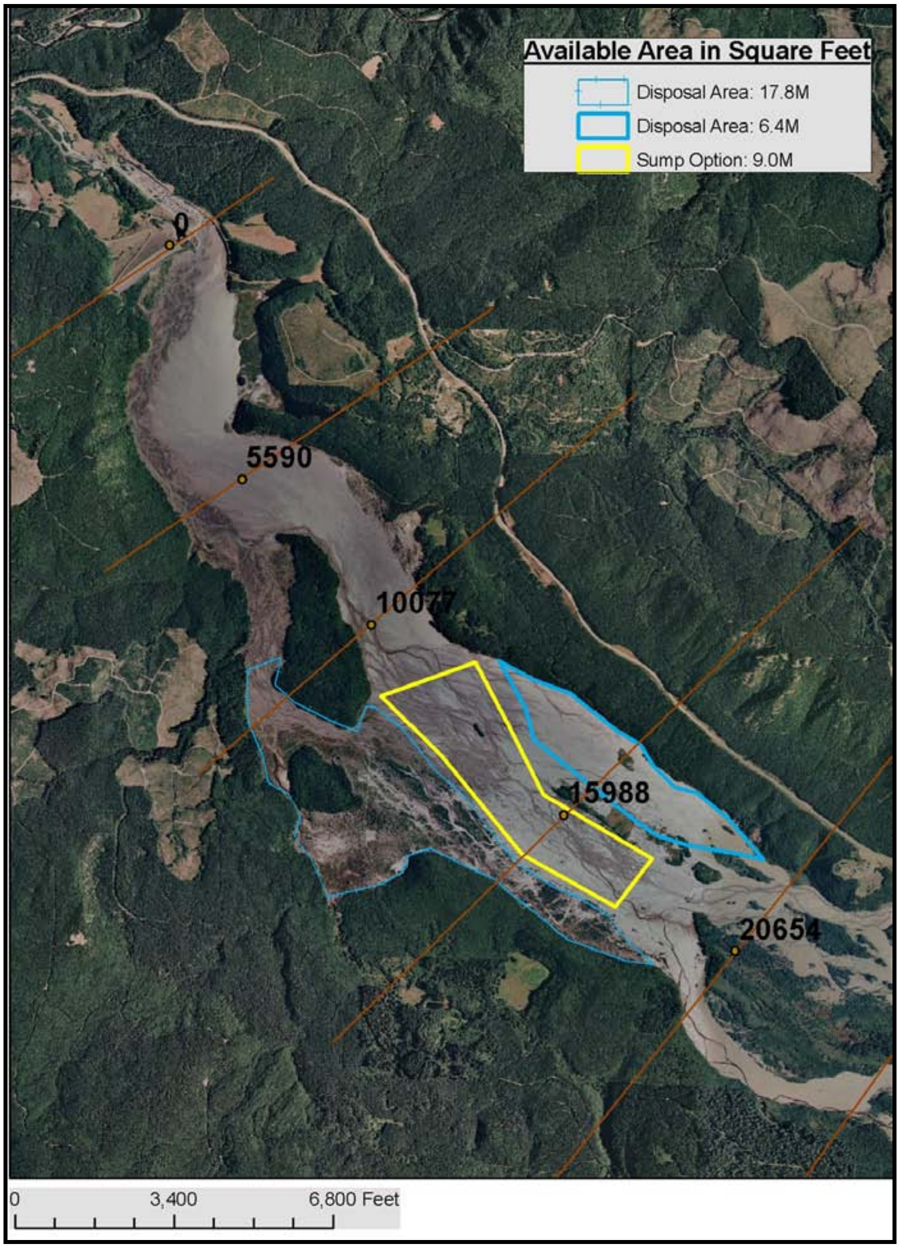
The Elk Rock Sediment Dam was not advanced for further consideration for two main reasons: (1) it violates the 1982 Monument Act, and (2) raising the SRS could accomplish the same amount of sediment storage for less cost.

9.4.3. Measure 4: Sediment Plain Sump

This measure involves operating a sump in the sediment plain above the SRS to trap sediment eroding off the debris avalanche. A potential sump location is shown in Figure 6. The volume of the sump would be 4 mcy (9 million square feet, 12-feet deep) and it would be allowed to fill with sediment from November through June. At the beginning of July, the river would be diverted to one side of the valley and the sump would be excavated mechanically in 4 months using scrapers or other equipment. The removed sediment would be stockpiled adjacent to the sump in the sediment plain, so as not to block any tributaries, as shown in Figure 6. From November 1980 to September 1981, 9.4 mcy of sediment was excavated from behind the N-1 structure. This excavation rate suggests the proposed excavation rate of 1 mcy/month is achievable.

The storage capacity within the sediment plain is limited. If the two disposal sites shown in Figure 6 are filled to a height of 40 feet, the storage capacity would be approximately 30 to 35 mcy. At a rate of 4 mcy per year, the sites would be full in 7 to 9 years. The disposal sites may require armoring or river redirecting structures on the sides adjacent the river to prevent the spoil material from eroding during high-flow events. The cost of such measures is estimated to be about $5 to $10 million. It is not certain, however, that the disposal sites would necessarily require stabilization.

Figure 6. Potential Sump Location on the Sediment Plain above the SRS



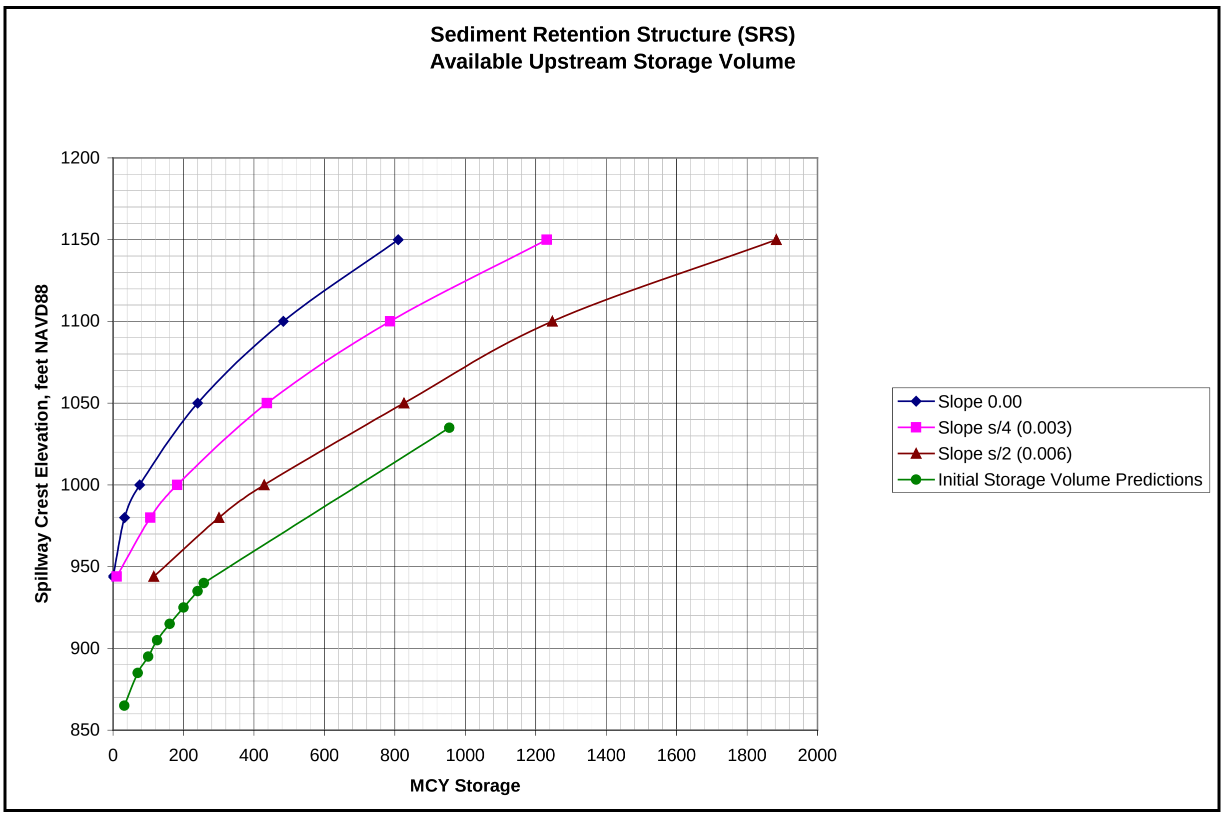
The cost to excavate the sediment from the sump is estimated to be approximately $20 million per year based on a unit cost of $5 per cy. For an 8-year period, the construction cost would be $160 million. Given the high cost and limited capacity, the sediment plain sump was not advanced for further consideration at this time. It may be considered later, however, if the understanding of conditions changes such that the sump measure becomes cost competitive.

9.4.4. Measure 6: Raised SRS Spillway

This measure involves raising the SRS spillway without raising the top elevation of the dam. The spillway crest could be raised by constructing a roller-compacted concrete (RCC) section. There is 60 feet of height between the existing spillway crest and the top of the dam. This is the height required to safely pass the operating basis mudflow (OBM), a mudflow resulting from an intra-episode eruptive event, roughly the size of the 1980 eruption, occurring at a time of maximum snowpack. The OBM is the largest mudflow considered realistic during the project life. The SRS was designed to withstand and pass the OBM without making downstream conditions worse. Under current conditions, there is an estimated 5 feet of freeboard available to pass the OBM. Any spillway raise without raising the top of the dam would reduce the ability of the SRS to safely pass the OBM.

Under current conditions with the sediment level at the spillway crest, the reservoir level is expected to rise 39 feet above the spillway crest during the probable maximum flood (PMF), leaving a freeboard of 21 feet. If the spillway were to be raised 20 feet, ignoring the OBM and nearly eliminating the freeboard during the PMF, the volume of sediment that would be stored up to a sediment slope of zero is about 20 mcy (see the curve for slope = 0 in Figure 7).

Figure 7. Raised SRS Sediment Storage Volume Calculations



Given the impacts to passing the OBM and PMF, raising the spillway without also raising the top of the dam would not be acceptable as a long-term measure. It may, however, be considered as a short-term measure if raising the entire SRS is part of the long-term plan. In this option, the spillway could be raised 20 feet in order to begin trapping more sediment as the design process for raising the SRS progressed. The initial spillway crest raise would ultimately be incorporated into the new raised spillway. This option may be considered if raising the entire SRS is part of the long-term plan.

9.4.5. Measure 7: Stabilization of Banks

This measure involves stabilization of banks on the mainstem Toutle, South Fork Toutle, and North Fork Toutle rivers not including banks within the debris avalanche. Some of the banks are dredge disposal sites from the 1980s. As discussed in the Sediment Budget Report (Corps 2009), 68 bank erosion sites were identified during the 2008 helicopter flight. The sites are shown in yellow in Figure 8. Bank erosion volumes were estimated by comparing changes in bank geometries from historic aerial photos from 1999 and 2006. Figure 8 shows the estimated volumes. The average bank length and height are 1,100 feet and 20 to 25 feet, respectively.

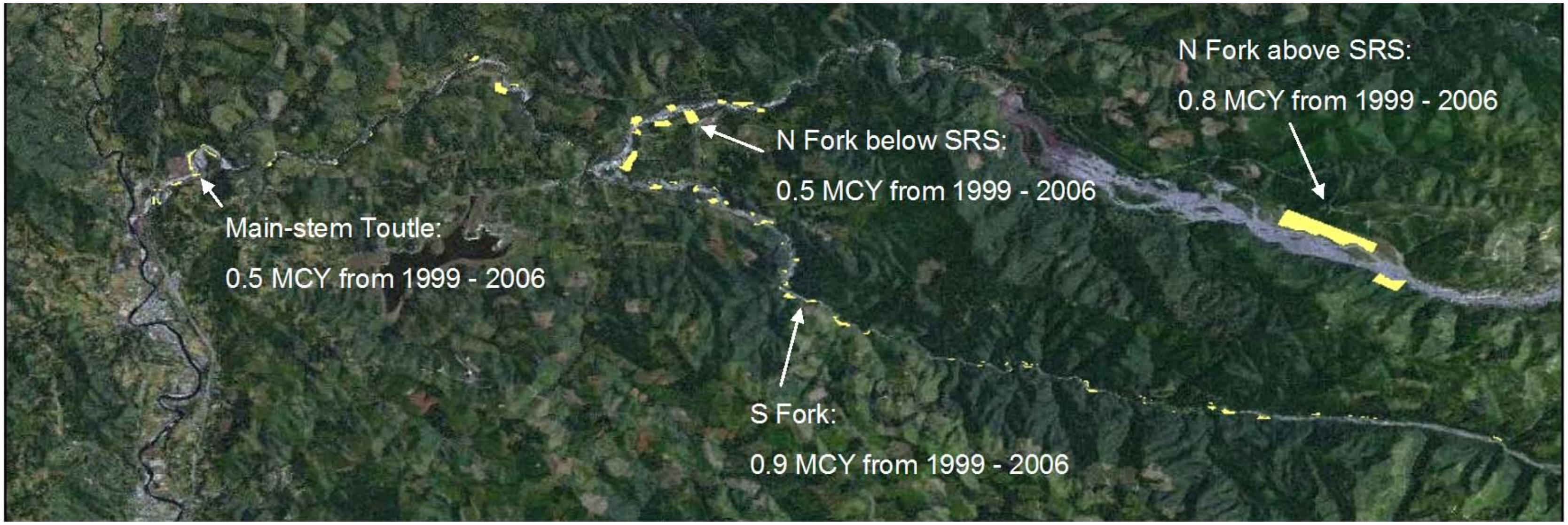
Compared to the debris avalanche sediment source, combined these banks are a small sediment source. The average bank erosion rate, over the time period from 1999 to 2006, from all 68 banks is 0.4 mcy per year. According to the Sediment Budget Report, this is only about 10% of the total sediment source. The debris avalanche contributes approximately 80% of the sediment.

The total length of all the bank sites is about 76,000 feet. Using an estimated stabilization cost range of $500 to $1,000 per foot of bank, the cost to stabilize all the banks would range from $38 million to $76 million. As the 68 identified banks are a small contributor to the overall sediment load, it is not considered worthwhile and cost effective to attempt to stabilize these banks as a general approach. However, some banks may be stabilized as a part of other measures. For example, the LT-1 (lower Toutle) dredge disposal banks would be protected from future erosion under the LT-1 sump measure.

9.4.6. Measure 9: Expand Floodplain on Toutle River

The concept of this measure was to expand the floodplain on the North Fork Toutle River below the SRS and on the lower Toutle, if possible, to provide flood storage volume and induce sediment deposition. However, the valley walls along these river segments are relatively steep and there is very little floodplain to potentially expand. There are no levees that could be set back. Sediment stabilization basins LT-1, LT-3, and NF-1 (North Fork Toutle) were operated in the early 1980s. In these basins, flow velocity was reduced causing sediment to deposit, after which the sediment was removed and placed in disposal areas adjacent to the river. It may be possible to excavate the dredge spoil piles in these three areas to expand the floodplain. The areas gained by this excavation would be small and would not have a big capacity to store flood water. This concept was not advanced due to the lack of floodplain on these river segments, making the concept not feasible.

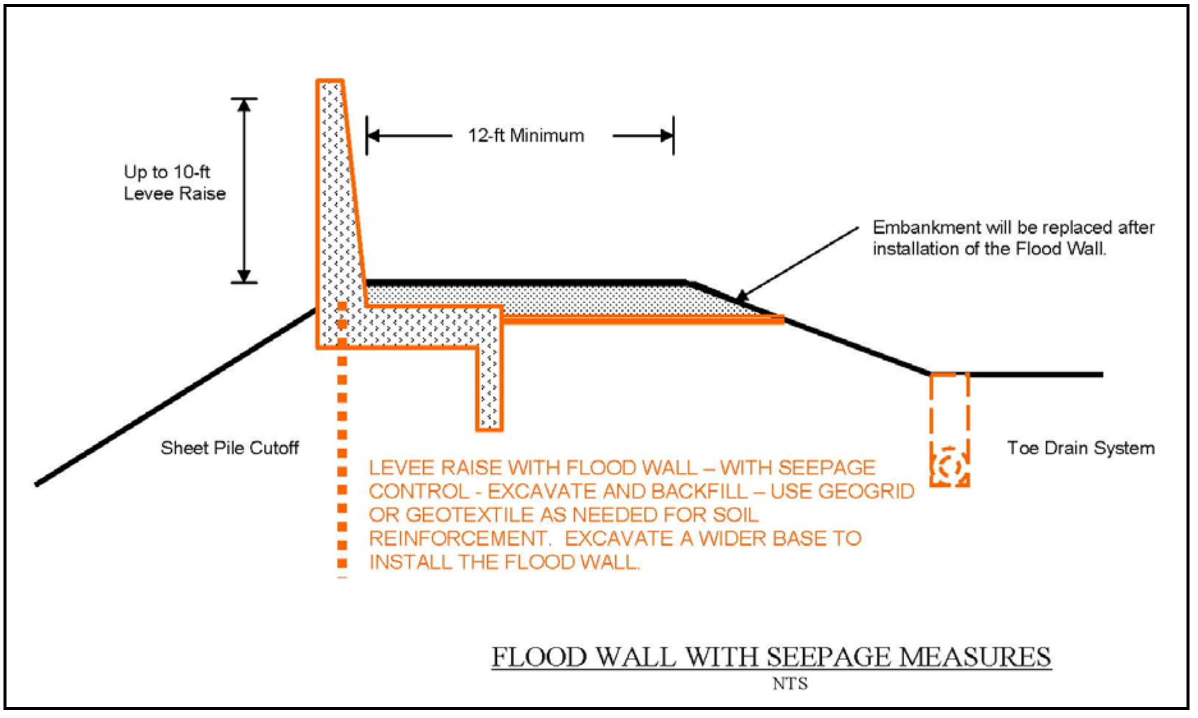
Figure 8. Potential Bank Stabilization Locations (not including debris avalanche)



9.4.7. Measure 11: Levee Improvements

This measure involves raising levees along the Cowlitz River. The highest raise considered is 10 feet. Two methods to accomplish levee raises would be to widen and raise the levee embankments or to construct floodwalls such as the one sketched in Figure 9. Raising the levee embankments would involve expanding the levee footprints, which would involve acquiring real estate on the interior side of the existing levees. To avoid the need to acquire new real estate, floodwalls with seepage control could be constructed.

Figure 9. Use of Floodwall to Raise Height of Levee



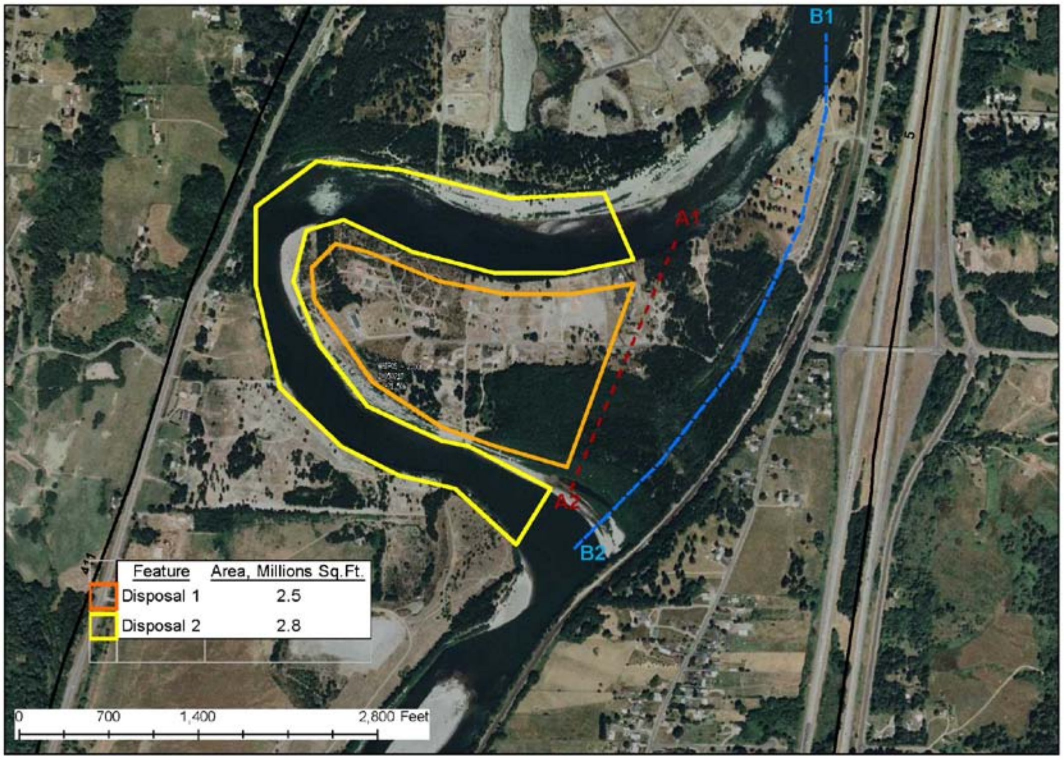
Raising levees would reduce the flood risk to the leveed areas in the near-term, and the effect could be extended by dredging or otherwise preventing continuing sedimentation in the river. However, there would be many negative impacts. If the river conveyance is not maintained, the non-leveed areas will suffer increased flooding. Behind the raised levees, the damages and threat to life caused by a potential levee failure would be higher as the depth of inundation would be greater. In addition, in order to raise the levees, some of the bridges crossing the river would have to be modified or raised.

The general approach of raising all levees was not advanced. There may be improvements to local areas that are appropriate, such as the installation of the seepage cutoff wall in the Castle Rock levee upstream of the bridge to increase the safe water level. After the no action modeling through 2035 is complete, the levee measure will be revisited. If flood profiles do not increase substantially, and if it appears possible to make significant impacts on levels of protection by working on limited portions of levees, the levee measure may be reconsidered.

9.4.8. Measure 14: Horseshoe Bend Sump or Cutoff

Horseshoe Bend is a meander in the Cowlitz River channel located between RM 12-14, downstream of Castle Rock and upstream of Lexington (Figure 10). The reach currently has a channel slope of approximately 0.04%. The existing point bar has been developed by private interests.

Figure 10. Horseshoe Bend Sump or Cutoff



Two concepts were considered for the Horseshoe Bend location. One concept was to create a sump at this location. The other concept was to cut off the oxbow, shortening the river to increase sediment transport and headcutting of deposited sediment upstream. To create a sump, the land on the inside of the bend would need to be acquired. Preliminary calculations indicate that there is not enough area to operate the sump and dispose of the dredged material for longer than a few years. After this time, the sediment removed from the sump would need to be hauled to another disposal site. Another limitation would be the short in-water work period: only the month of August. For these reasons (lack of space and time), the sump concept was not evaluated further.

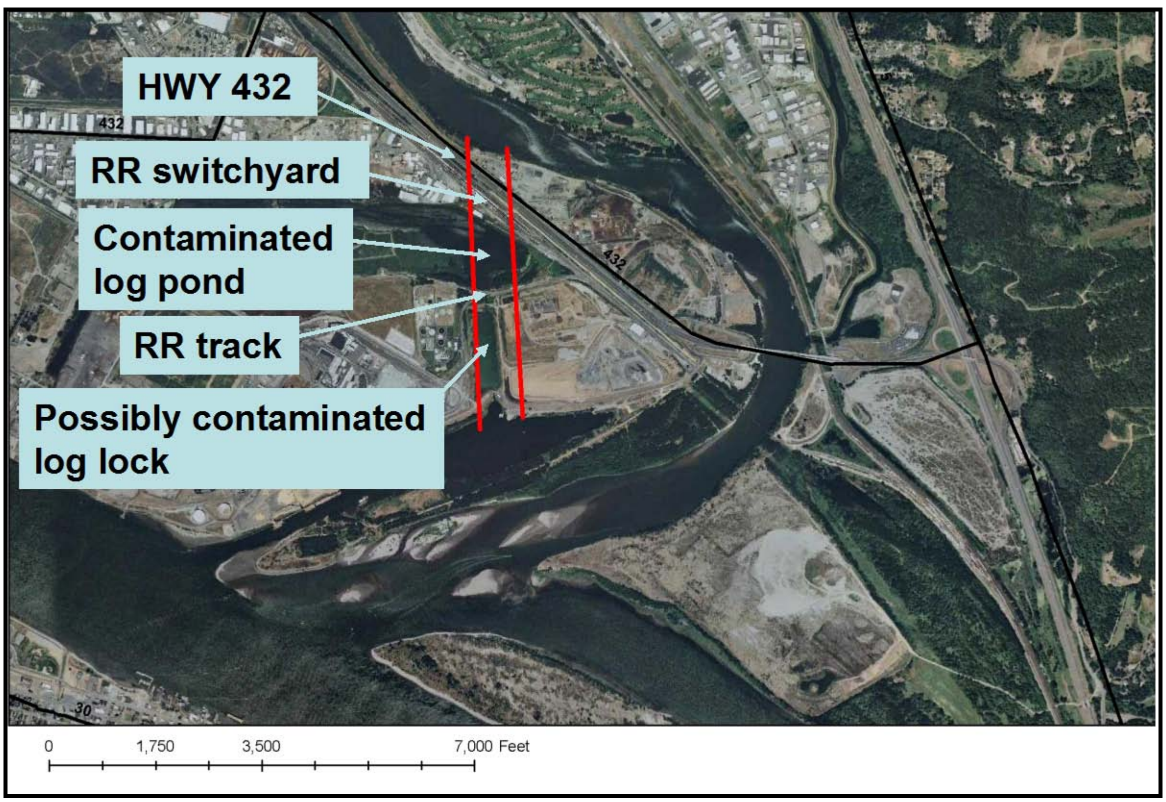
Two methods were considered for creating a cutoff channel. In both methods, the roughly 400-foot-wide cutoff would be excavated along an alignment such as A1-A2 or B1-B2 in Figure 10. In the first method, excavated material from the new channel would be used to plug the entrance to the existing meander. In the second method, sediment entrainment structures such as dikes would be placed to cause the deposition of sediment within the existing meander. As sedimentation occurred, the flow would abandon the existing meander and move into the newly excavated channel. The new cutoff would increase the channel slope from 0.04% to a range of 0.07% to 0.13%. As with the sump option, some land would need to be acquired to implement this measure.

During level one screening, the effectiveness of cutting off Horseshoe Bend on sediment transport and upstream headcutting was not evaluated, but was thought to be likely minor in terms of reducing flood risk for Castle Rock, Lexington, Longview, and Kelso. It was decided to not investigate the measure further unless the following situation occurs. If the other measures related to flushing sediment or increasing sediment transport – releasing flushing flows from Mossyrock Dam and installing dike fields at the mouth of the Cowlitz – proved marginally effective, the cutoff at Horseshoe Bend would be evaluated to determine the incremental effect.

9.4.9. Measure 15: Reconnect Old Channel near Mouth of Cowlitz

Downstream of RM 1.0, the confined Cowlitz River channel broadens from approximately 600 to 700 feet to nearly 1,500 feet across. Deposition of sediment is a problem in this area. This measure proposes relocating the river back to an earlier, shorter alignment, as shown in Figure 11, to enhance sediment transport and induce headcutting of deposited sediment upstream. This measure was not advanced due to the significant industrial/commercial sites and infrastructure within the proposed re-alignment and the potential for exposing contaminants during excavation. Figure 11 shows some of the major features within the re-alignment area.

Figure 11. Potential Re-alignment of Cowlitz River near the Mouth



9.5. SECOND SCREENING

9.5.1. Planning Horizons

Currently, the average sediment yield rate from the debris avalanche is assumed to range from 5 to 10 mcy per year for the foreseeable future. In any given year, the yield may be much lower or higher; the 5 to 10 mcy per year is an estimated *average* rate. From the sediment budget, the average sediment yield is 6 mcy per year. The assumption of a steady average yield rate is considered valid until the current study of the debris avalanche sediment yield rate decay outlook is complete at the end of 2010.

To date, average yield rates ranging from 5 to 10 mcy per year have been used to evaluate measures; the sediment budget, reporting 6 mcy per year, has been used in development. From now on, an average yield rate of 6 mcy per year will be used for consistency.

The measures in the second screening were evaluated for two planning horizons: 25 years (to year 2035) and 50 years (to year 2060). The first horizon matches the congressionally authorized project life. The analysis was also performed for the longer second horizon to evaluate the impact to the decision process for the possible condition of quasi-steady-state erosion from the debris avalanche beyond the congressionally authorized project life.

9.5.2. Measure 3 - Grade Building Structures

General Description of Strategy

Grade building structures (GBS) would be built in the sediment plain above the SRS for the purpose of increasing sediment deposition in the plain. After construction in the late 1980s, the SRS provided a sediment trapping efficiency of approximately 92%. In 1998, the sediment level behind the SRS reached the elevation of the spillway and the project has since been run-of-river, with all flow passing the spillway. In the run-of-river condition, more sediment is passing the SRS and the trapping efficiency has dropped to approximately 31%. The goal of constructing GBSs in the sediment plain would be to increase the trapping efficiency of the SRS/sediment plain system.

Implementation Approach

Several concepts have been considered for GBSs including valley-spanning grade control structures, groins originating from alternating sides of the valley, and structures built to seed the formation of islands in the sediment plain. Valley-spanning grade control structures would be low-height (3-15 feet) dams with spillways. The dams would create pools that would allow for the deposition of sediment. Groins originating from alternating sides of the valley, extending approximately two-thirds across the valley, would increase the length of the river, thereby decreasing its slope and increasing the tendency for deposition. During high-flow events, the groins also may increase the pooling of water which would increase sediment deposition. Structures such as engineered log jams, pile dikes, or interlocking concrete armor units (e.g., “A-Jacks”), potentially in combination with vegetation plantings, could be built at several locations to add large-scale roughness to the sediment plain. The goal would be for islands of sediment to form around the structures.

The GBS measure is envisioned as an adaptive management approach. Construction would not be limited to one season. Over time, as sediment deposits around the structures, new structures would be built as needed to continue the sediment trapping and the "building of grade" in the sediment plain.

The expected performance of GBSs is unknown. Lessons from emergency measure activities in the early 1980s, such as N-1, indicate that sediment can be trapped but that such structures can be overwhelmed and fail due to extreme events. For the GBS measure, how much sediment would the structures trap? What features need to be incorporated so that the structures can resist the forces of the river without failing, or if they fail, what are the consequences? In order to understand the potential for sediment trapping and stabilization that the GBSs measure may provide, a pilot project is proposed.

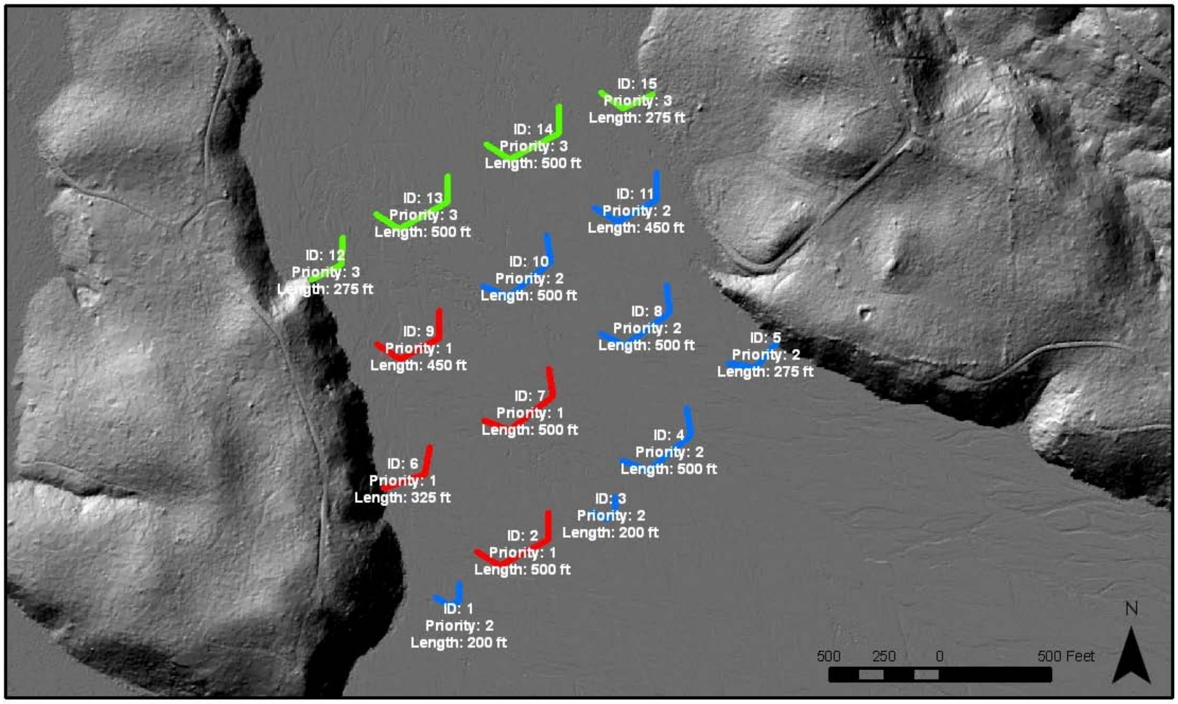
The purpose of the pilot project is to test the ability to build GBSs and to test the performance of the GBSs in terms of primarily sediment retention and durability. The plan is to build and test a variety of GBS types. The observed performance of the GBSs will be used to evaluate the potential use of GBSs as a long-term measure for sediment management. Two potential outcomes of the pilot project have been identified.

1. If the pilot project GBSs trap sediment, and other long-term analyses indicate that only a moderate increase in trapping efficiency is required at/above the SRS (e.g., a trapping efficiency of 50% would provide useful sediment management), then GBSs will continue to be evaluated as a tool for long-term sediment management. In this long-term scenario, GBSs could be constructed as needed on the sediment plain to maintain the required trapping efficiency. Spikes in sediment transport that overwhelm the GBSs would be managed by downstream measures such as dredging in the Cowlitz River.

2. If the pilot project GBSs trap little or no sediment, or other long-term analyses indicate that a trapping efficiency greater than that the GBSs can provide is required at/above the SRS (e.g., a trapping efficiency of 80%), then GBSs may not be considered as a long-term tool.

Figure 12 is an example that provides an idea of the GBS concept for the pilot project. In this example, the structures are all timber pile dikes. The direction of flow is north. The design of the pilot project grade building structures is currently underway. Also, in coordination with the pilot project, a concurrent study is being performed to optimize and model the potential long-term application of grade building structures in the sediment plain.

Figure 12. Example Concept for Grade Building Structure Alignments



Cost Estimate

A cost estimate has not been made for the GBSs measure due to uncertainties in the expected performance of the structures and thus, the expected long-term implementation of the structures. The results of the pilot project will aid in estimating the costs of the GBSs measure. Because the measure is adaptive, the cost estimate will involve construction costs spread out over the planning horizon rather than a large one-time construction cost.

Environmental Considerations

Of the three GBS approaches described above – grade control structures, groins, and island-forming structures – the grade control structures would have the most negative impact on fish. Fishways, the expected reliability of which is questionable, would need to be constructed for upstream fish passage. In addition, the shallow pools behind the structures may cause fish passage problems. The groins and island-forming structures approaches would not restrict fish passage and the structures could be located to reduce interference with fish access to tributaries.

Real Estate Considerations

For the planning horizon through 2035, no additional real estate needs are expected, as the GBSs and associated sediment storage would remain within the sediment plain boundary identified for SRS sediment storage. If the planning horizon is much longer, the real estate issue will need to be revisited.

Discussion

The adaptive-management GBS measure has the potential advantage of storing moderate volumes of sediment above the SRS with low impact to the environment. The main disadvantage at this time is the uncertainty in the expected performance of the measure. A pilot project is proposed to reduce this uncertainty.

9.5.3. Measure 5 - Raised SRS Dam and Spillway

General Description of Strategy

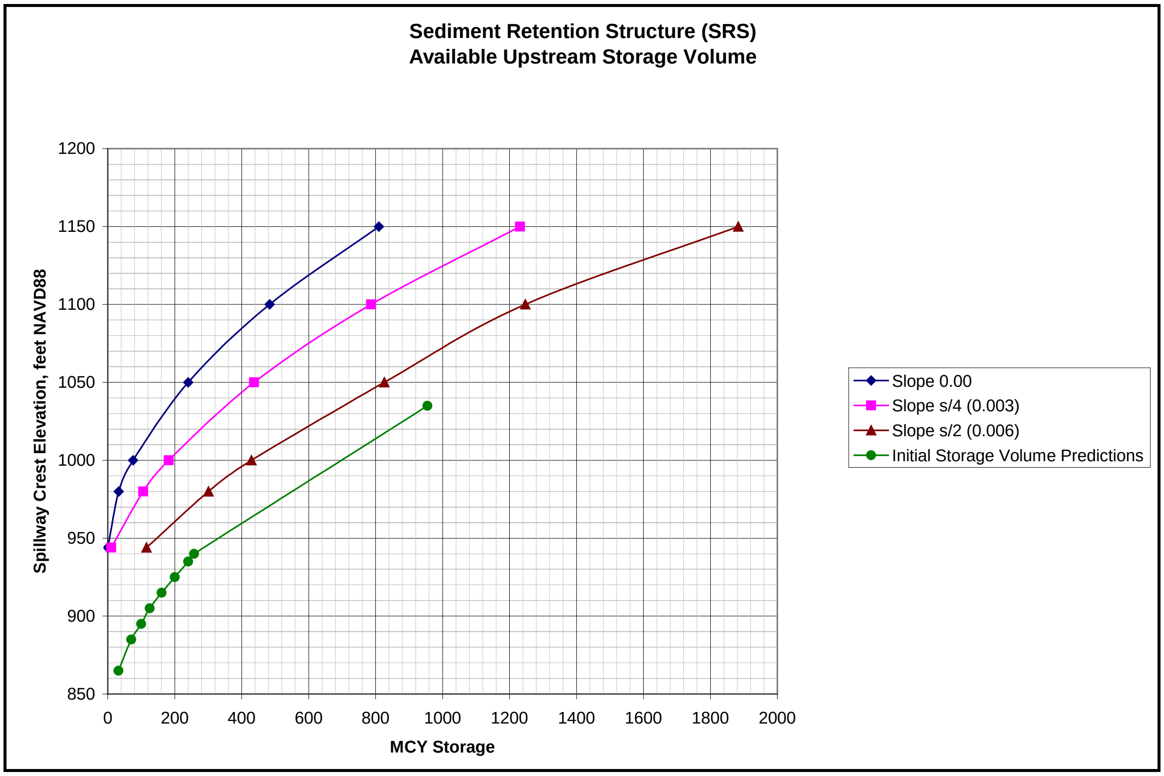
As discussed under Measure 3, the existing SRS was designed for an ultimate sediment slope of 0.006 (S/2). With this slope, the sediment storage volume would total 258 mcy. Under current conditions, with a fairly flat slope directly behind the SRS, the volume of sediment trapped to date is over 100 mcy. The SRS is now in sediment retention Phase II, where gravel and sand deposits behind the dam and some sand passes the SRS, as was expected during design of the structure. It is possible to raise the SRS so that the structure could again operate as Phase I with sediment depositing behind the structure. Similar to when the existing SRS was in Phase I, flow would pass the raised structure through a set of outlet works pipes. In the future, the sediment level would again reach the crest of the new spillway and the structure would operate in Phase II.

During the planning of the existing SRS in the 1980s, consideration was given to building the SRS so that it could be raised in the future. Due to cost constraints, however, the SRS was not built to accommodate a large raise. With the current dam embankment crest width of 60 feet, upstream slope of 2.5H:1V and downstream slope of 3H:1V, the dam embankment could be easily raised only 7 feet if the new crest width is 20 feet and the upstream and downstream slopes remain the same. Other approaches, such as use of steeper, reinforced slopes, may be considered to gain more than 7 feet in height. The SRS raise concept described below is a new concept not originally included in the 1980s planning and design.

Implementation Approach

For the timeframe through 2035, the approximate volume of sediment storage capacity required would be roughly 200 mcy if (1) the raised SRS were to capture all sediment from the debris avalanche, i.e., the raised SRS had a trapping efficiency of 100% (for reference, the existing SRS had a trapping efficiency of 92% when all flow passed through the outlet works), and (2) the sediment load from the debris avalanche is 8 mcy per year for 25 years. In further studies, a lesser trapping efficiency consistent with the Cowlitz River's transport capacity will be evaluated, but for this Progress Report the overly conservative approach of providing storage capacity for all the debris avalanche erosion was considered (in addition, future evaluations will use an average annual sediment load of 6 mcy per year rather than 8). To provide the capacity to trap this volume with a sediment slope of zero behind the spillway crest, the spillway would need to be raised 100 feet, to a new elevation of 1,040 feet, as determined from Figure 13. The new top of dam elevation would be 1,100 feet.

Figure 13. Raised SRS Sediment Storage Volume Calculations



For the longer timeframe to year 2060, it was decided to develop a conceptual design for a raise of 160 feet. This raise, with a new spillway elevation of 1,100 feet, results in almost 500 mcy of storage with a sediment slope of zero behind the spillway crest (this is an overly conservative volume estimate; future evaluations will target a smaller volume to trap). The new top-of-dam elevation would be 1,160 feet. If the dam were to be raised above this elevation, Highway 504 on the right abutment would need to be relocated.

Future work for the raised SRS measure involves the following five concepts, all of which are smaller than the raises described in this Progress Report:

- **30 feet raise.** New spillway crest elevation of 970 feet and new top of dam elevation of 1,030 feet. This raise will provide an additional storage capacity of about 60 mcy up to a sediment slope of about 0.003 behind the spillway crest.

- **40 feet raise.** New spillway crest elevation of 980 feet and new top of dam elevation of 1,020 feet. This raise will provide an additional storage capacity of from 35 to 110 mcy up to a sediment slope range from 0 to 0.003 behind the spillway crest (the freeboard for this concept is only 40 feet compared to 60 feet for the other concepts).

- **50 feet raise.** New spillway crest elevation of 990 feet and new top of dam elevation of 1,050 feet. This raise will provide an additional storage capacity of about 60 mcy up to a sediment slope of zero behind the spillway crest.

- **50 feet raise adaptable to 70 feet raise.** A structure shall be designed that would first be built up 50 feet (spillway elevation 990 feet and top of dam elevation 1,050 feet) and then would allow for future adaptation to 70 feet (spillway elevation 1,010 feet and top of dam elevation 1,070 feet) to accommodate continued sediment load from the debris avalanche. It is expected that the 50 feet raise part of the adaptable design would be more expensive to construct than the non-adaptable 50 feet raise described above.

- **70 feet raise**. New spillway crest elevation of 1,010 feet and new top of dam elevation of 1,070 feet. This raise will provide an additional storage capacity of about 120 mcy up to a sediment slope of zero behind the spillway crest.

For this Progress Report, a conceptual design was developed for the larger raise of 160 feet. This is an upper bound to how high the SRS could be reasonably considered to be raised; higher raises would require Highway 504 on the right abutment to be relocated. In order to make a preliminary cost estimate for raising the SRS by 100 feet, it was assumed that the same features designed for the larger raise would be built, but would be proportionally smaller. Costs for these features were scaled down by 100/160, the ratio of the raises.

Five different configurations were considered for raising the SRS:

1. Maintain same dam axis; new outlet works separate from new spillway.
2. Maintain same dam axis; new outlet works incorporated into new spillway.
3. Maintain same dam axis; new outlet works built in existing spillway; new spillway excavated in rock in right abutment.
4. Shift new dam axis downstream; new outlet works separate from new spillway.
5. Shift new dam axis downstream; new outlet works incorporated into new spillway.

The advantage of shifting the dam axis downstream is that the new upstream slope would not be constructed over the sediment plain, a liquefiable material. The disadvantage is that the new outlet works and spillway crest would also shift downstream, which could make design and construction of these features more difficult. For the purpose of this analysis, it was decided to maintain the existing dam axis and improve the sediment foundation as described below. The idea of incorporating the new outlet works into the new spillway was explored but, during the time period of this analysis, a workable configuration was not identified. The plan below maintains the outlet works separate from the spillway. This is configuration 1 in the list above. In addition, as time allowed, configuration 3 in the list above was evaluated to see how much cost savings might be obtained by reducing new concrete volumes.

Figures 14 to 17 show the conceptual design for raising the SRS dam and spillway 160 feet (configuration 1). The main features of the design include: (1) *in situ* densification of the sediment plain to support the upstream part of the new embankment; (2) a new embankment section over the existing embankment section, extending up the left abutment; (3) a new retaining wall to contain the embankment adjacent to the outlet works; (4) a new outlet works over the existing outlet works; (5) a new spillway section over the existing spillway; and (6) a new roller-compacted concrete (RCC) section on the right abutment.

Figure 14. Plan for Existing SRS

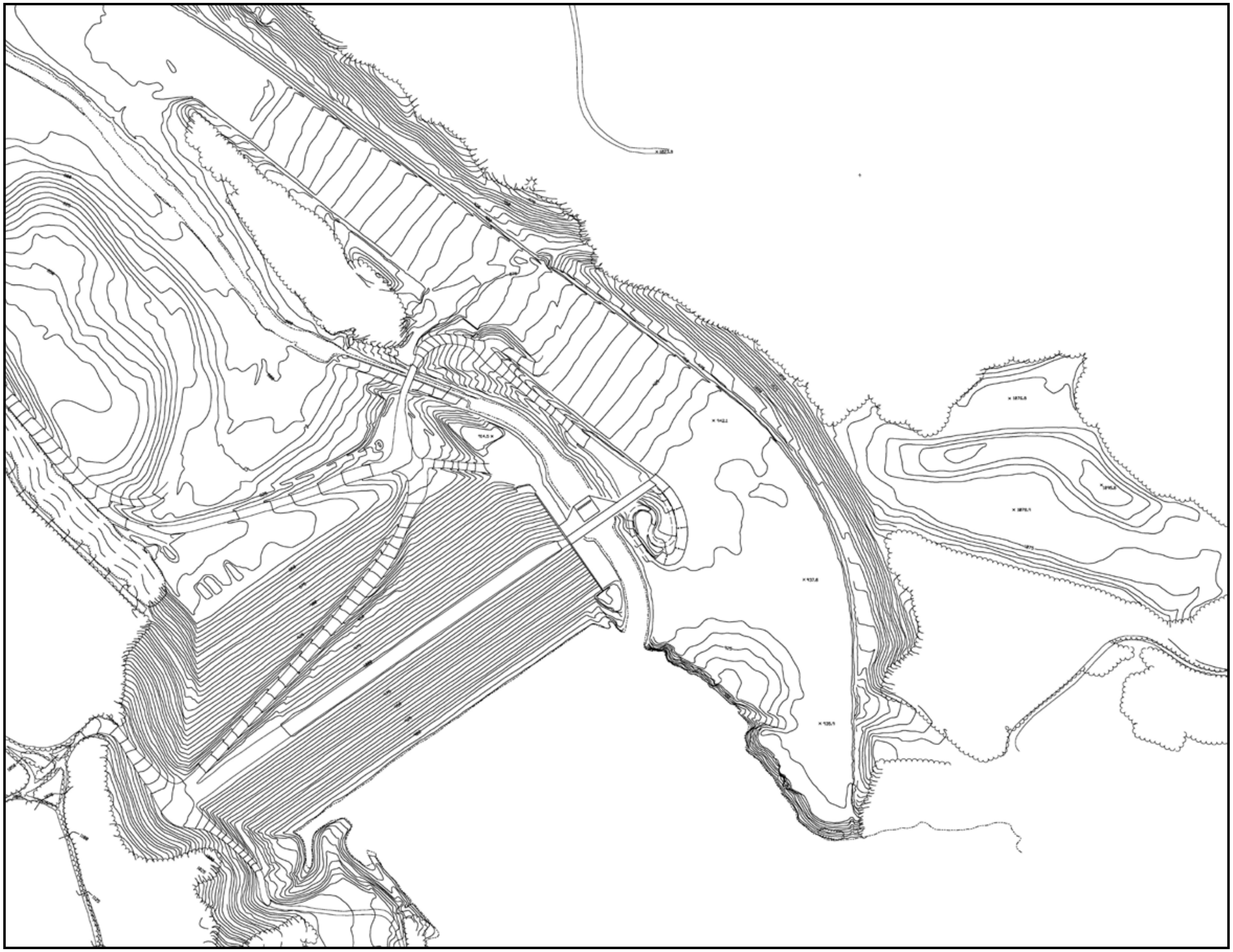


Figure 15. Plan of Potential SRS Raised 160 feet, Configuration 1

Figure 16. Section of Potential SRS Embankment Raised 160 feet, Configuration 1

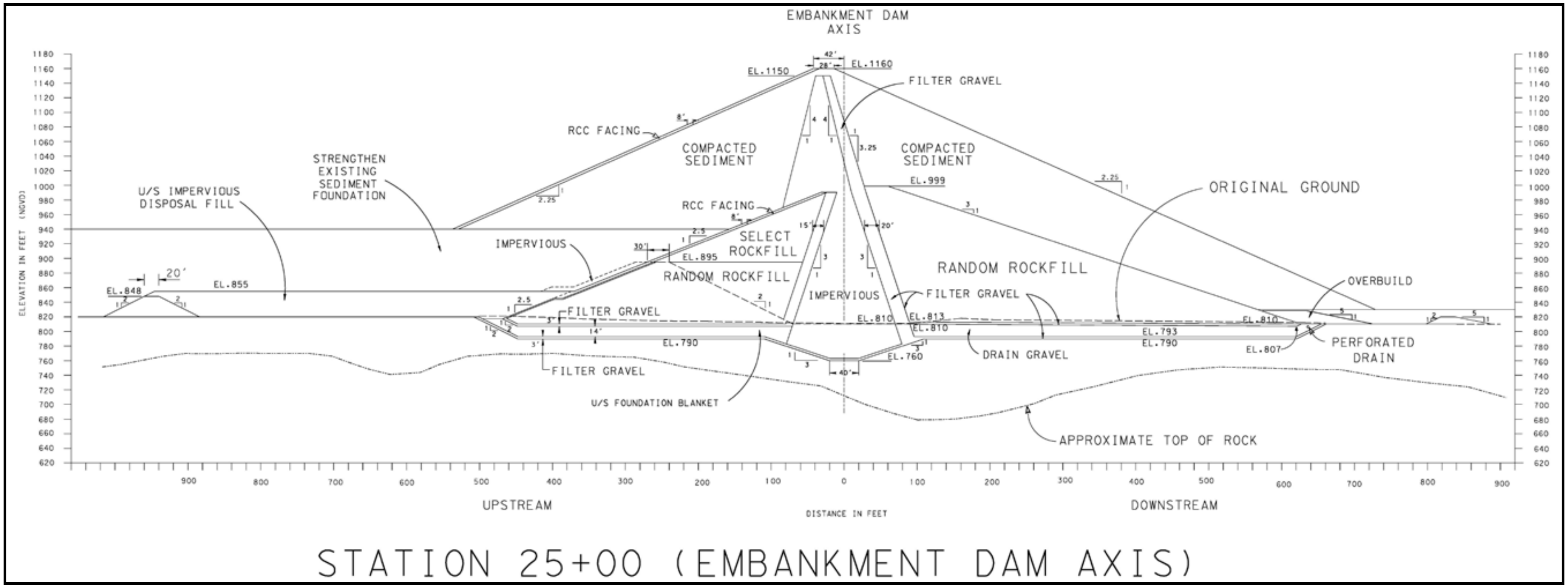
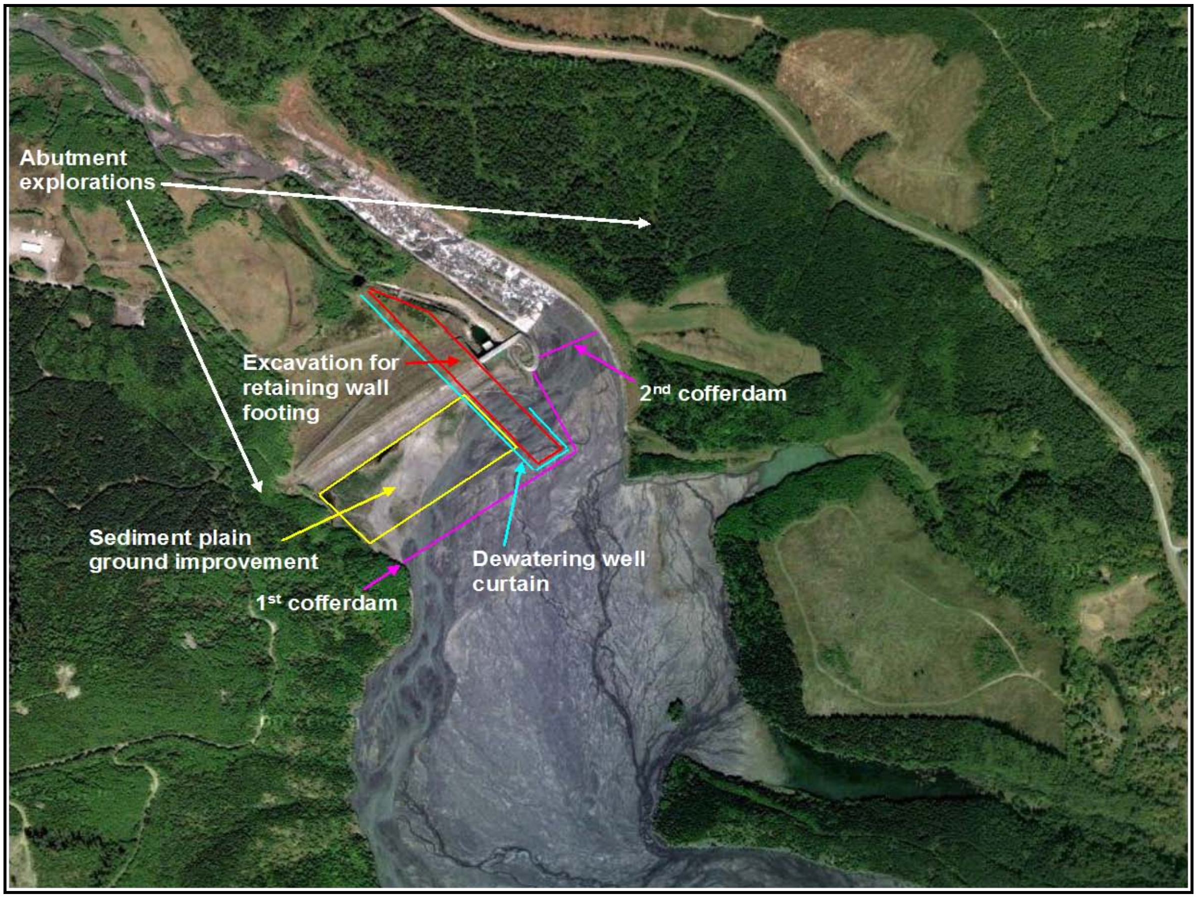


Figure 17. Earthwork, Dewatering and Cofferdams for SRS Raised 160 feet, Configuration 1



River diversion for construction would be accomplished using cofferdams (see Figure 17). The first cofferdam would be constructed upstream of the dam from the left valley wall to the spillway approach pier, with flow over the spillway. This first cofferdam would enable the sediment plain densification program and raising of the embankment dam, retaining wall, and outlet works. When that work is complete a second cofferdam would be constructed from the spillway approach pier to the right valley wall, with flow diverted through the outlet works. The second cofferdam would enable construction of the raised spillway. The cofferdams would be constructed of sediment from the sediment plain and would be armored on the upstream slope. The height of the cofferdams would be approximately 12 feet above the sediment plain elevation.

The existing very loose sediment against the SRS would be densified *in situ* in order to support the new embankment and reduce the material's liquefaction potential. The surface area and volume of treatment would be about 700,000 square feet and 1.8 mcy. The area is shown in Figure 17. The maximum depth of treatment would be about 85 feet, from approximately elevation 940 feet down to elevation 855 feet, which is the top of the existing upstream impervious disposal fill. One method to densify the generally silty sand sediment to a relative density of 70% would be vibro-compaction using one probe per 70 square feet of surface area. Assuming two rigs, each able to probe 1,500 feet per day, the ground improvement program would take about 7 to 8 months. Pre and post *in situ* density measurements using a cone penetrometer test would be performed to help design the densification program and verify performance.

Figure 16 shows the raised embankment section. About 30 feet of overburden would be removed from the left abutment and the underlying rock would be treated to support the new embankment. The excavated overburden would be used for the new core material. The shells would be constructed of sediment from the sediment plain. Compaction of this material would be critical to prevent strength loss during a potential seismic event.

As the sediment is mostly sand, it is assumed that filter criteria would be met between the compacted sediment and the core. The filter gravel chimney drain would be extended up into the raised embankment to collect seepage through the core. Material for the filter gravel would be obtained from the coarser parts of the sediment plain. To protect the upstream slope from scour, the RCC facing would be extended to the top of the raised dam.

Figure 18 shows the raised outlet works. The section includes fifteen new rows of outlet pipes, with the lowest starting at elevation 950 feet and the highest starting at elevation 1,090 feet. The rows would be 10 feet apart vertically. Each row would include five pipes. The major difference between the existing and new outlet works is that the new configuration would not discharge the flow into a free fall like the existing one did, as this is believed to harm juvenile fish passing through the structure. Instead, the pipes would discharge into a channel with weirs. The weirs are designed to break up the energy of the flow for downstream fish passage. A minimum depth would be provided, and the main channel would slope to a low flow channel on one side of the main channel. As with the existing outlet works, the new outlet pipes would be closed as the sediment level behind the dam comes up, until eventually all flow would pass the spillway.

Figure 19 shows the raised spillway. Whereas the existing spillway was cut into rock, this spillway is built up using RCC and concrete. Due to the steeper slope, weirs are required to provide downstream fish passage. The weirs are designed to break up the energy of the flow. A minimum depth would be provided, and the main channel would slope from each side to a low flow channel in the center. Note the need for the concrete wall on the left side of the spillway to contain high flows. On the right side of the spillway, overburden soil would be removed and a RCC wall would be built on bedrock to contain flows. The RCC section would extend northeast to close the dam on the right abutment.

Figure 18. Section and Plan of Potential SRS Outlet Works Raised 160 feet, Configuration 1

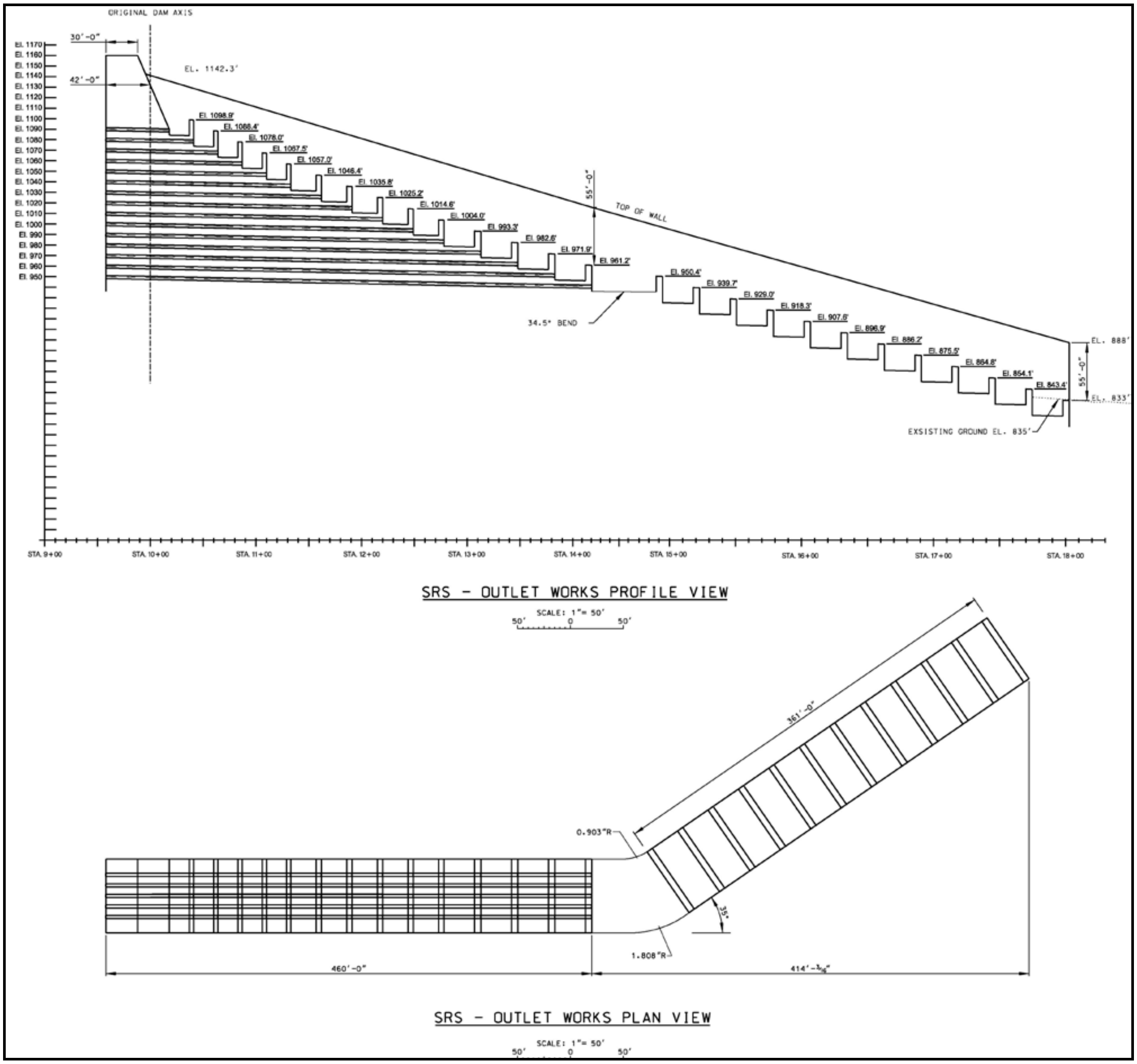
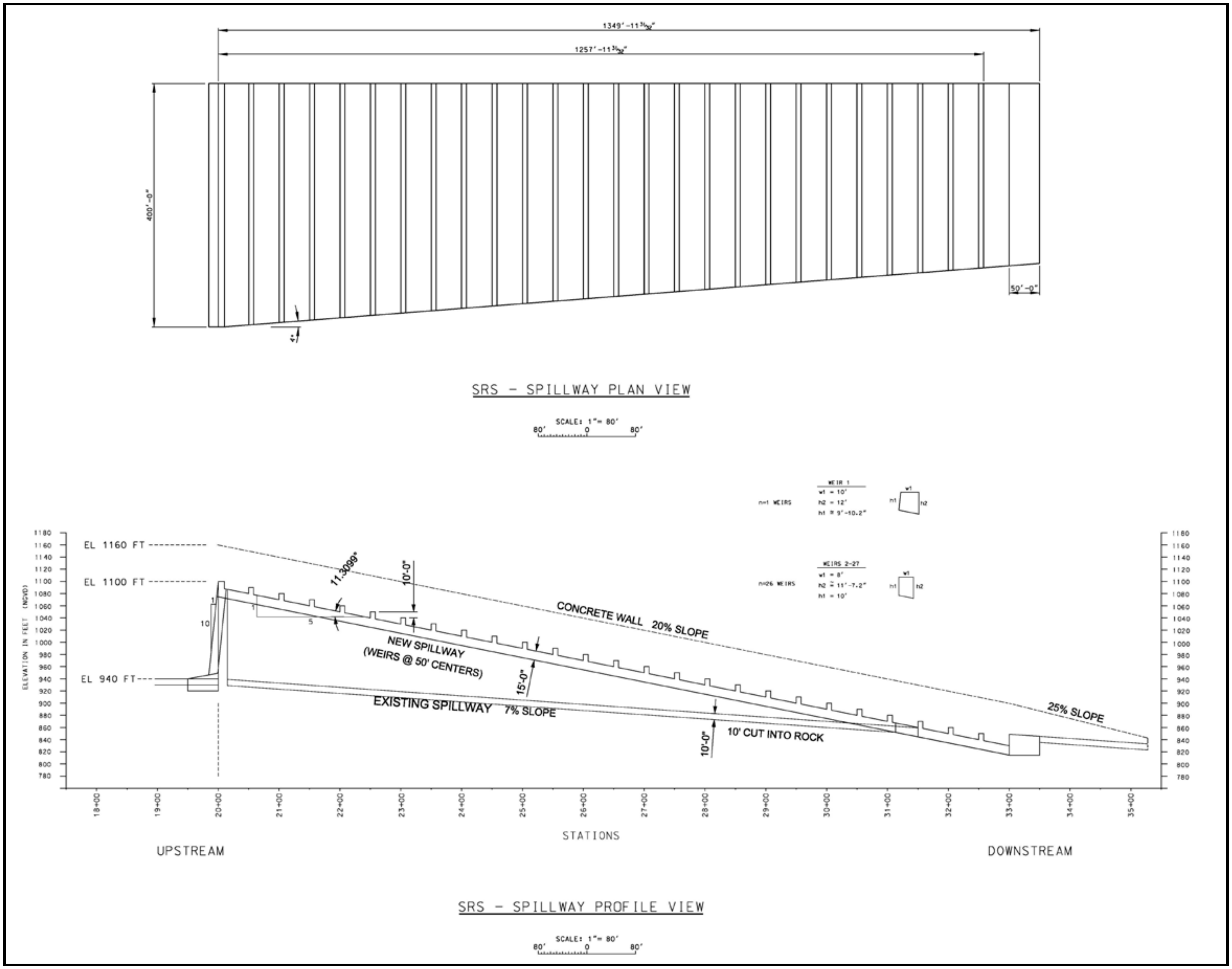


Figure 19. Section and Plan of Potential SRS Spillway Raised 160 feet, Configuration 1



Cost Estimates

Table 4 shows the cost estimate for the conceptual design of configuration 1. The cost estimate of $760 million is dominated by the large volumes of concrete required. Figures 20 and 21 show an alternative design, configuration 3, in which the new spillway is cut into rock farther up on the right abutment and the new outlet works is built within the existing spillway. While this configuration adds rock excavation volume and cost, a greater cost reduction is realized by reducing concrete volumes. As a result, the cost estimate of configuration 3 is $610 million, as shown in Table 5.

In order to make a rough estimate of cost for raising the SRS by 100 feet, the smaller raise for the timeframe through 2035, it was assumed that the same features designed for the larger raise of 160 feet would be built, but would be proportionally smaller. All costs except for the cofferdam costs were scaled down by 100/160, the ratio of the raises. The costs for the 100-foot raise are $480 million and $380 million for configurations 1 and 3, respectively.

Table 4. Cost Estimates for SRS Raise Configuration 1 – 160 feet and 100 feet Raise

SRS raised 160 ft Storage capacity 500 mcy					SRS raised 100 ft Storage capacity 200 mcy
Item	Quantity	Unit	Unit cost	Cost	Cost*
Abutment explorations	1	job		$1,000,000	$630,000
Mob/demob	1	job		$6,400,000	$4,000,000
Cofferdams	1	job		$1,500,000	$1,500,000
Sediment plain ground improvement					
Vibro-compaction	694,571	LF	$4.90	$3,400,000	$2,100,000
Pre- and post-in situ tests	1	job		$1,000,000	$630,000
Dewatering	1	job		$3,000,000	$1,900,000
Excavation					
Abutment overburden	629,962	CY	$8.11	$5,100,000	$3,200,000
Existing embankment and foundation for retaining wall footing	1,500,000	CY	$7.98	$12,000,000	$7,500,000
Embankment					
Replace embankment and foundation excavated for retaining wall footing	1,500,000	CY	$1.65	$2,500,000	$1,500,000
New core	629,962	CY	$1.36	$860,000	$540,000
New filter gravel, including excavation and processing from sediment plain	178,784	CY	$4.79	$860,000	$540,000
New compacted sediment "shells," including excavation from sediment plain	7,791,962	CY	$3.81	$30,000,000	$19,000,000
Roller-compacted concrete (RCC)	4,459,498	CY	$70.20	$310,000,000	$200,000,000
Mass concrete	680,906	CY	$185.94	$130,000,000	$79,000,000
Structural concrete	880,389	CY	$288.03	$250,000,000	$160,000,000
Total				$760,000,000	$480,000,000

* Quantities and costs were estimated for the 160 ft raise. 100 ft raise option would include same features; all costs except cofferdams scaled by height of raise.

Figure 20. New Spillway Cut in Rock for SRS Raised 160 feet, Configuration 3

Figure 21. New Outlet Works Location for SRS Raised 160 feet, Configuration 3

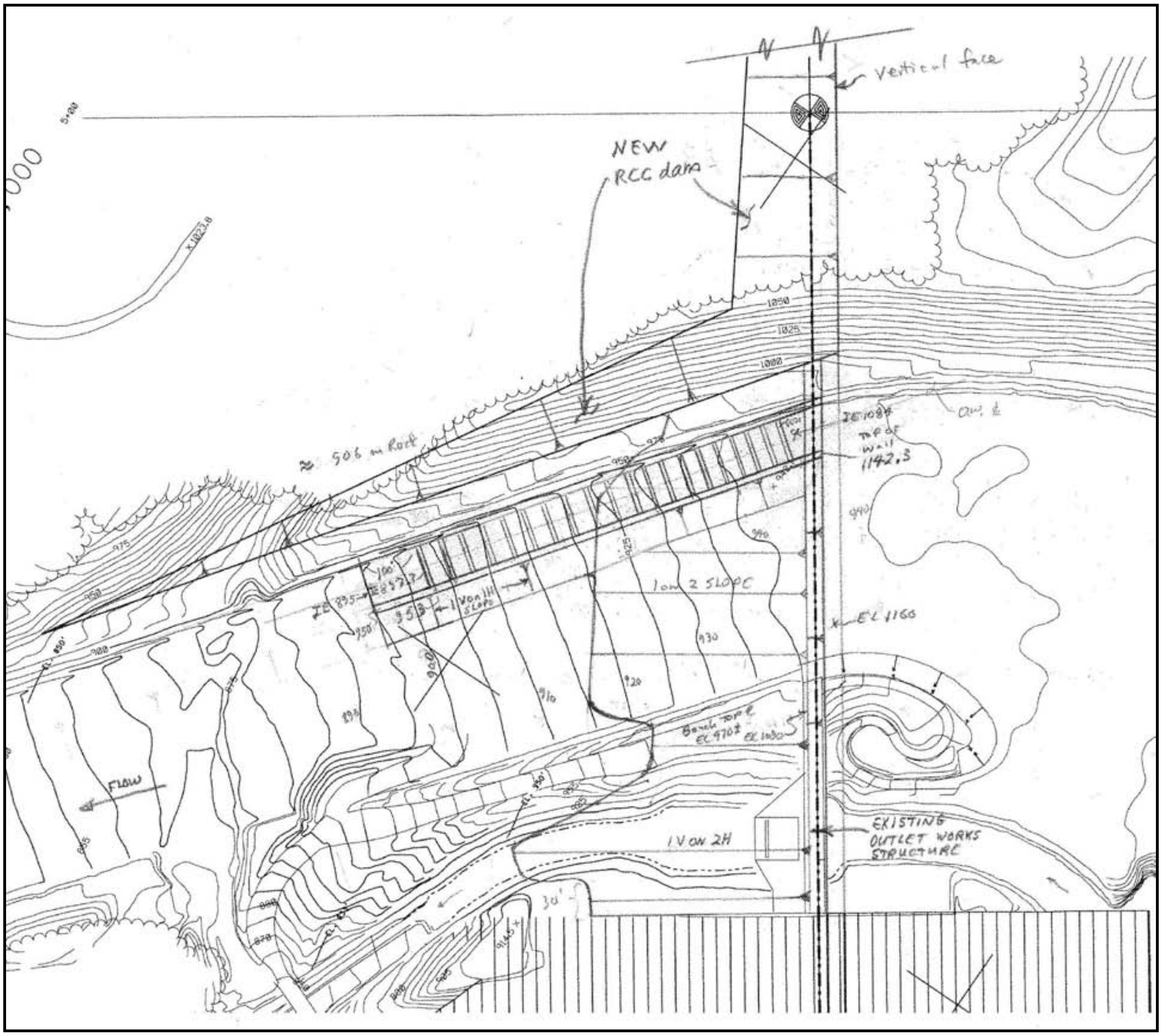


Table 5. Cost Estimates for SRS Raise Configuration 3 – 160 feet and 100 feet Raise

SRS raised 160 ft Storage capacity 500 mcy					SRS raised 100 ft Storage capacity 200 mcy
Item	Quantity	Unit	Unit cost	Cost	Cost*
Abutment explorations	1	job		$1,000,000	$630,000
Mob/demob	1	job		$6,400,000	$4,000,000
Cofferdams	1	job		$1,500,000	$1,500,000
Sediment plain ground improvement					
Vibro-compaction	694,571	LF	$4.90	$3,400,000	$2,100,000
Pre- and post-in situ tests	1	job		$1,000,000	$630,000
Dewatering	1	job		$3,000,000	$1,900,000
Excavation					
Abutment overburden	629,962	CY	$8.11	$5,100,000	$3,200,000
Existing embankment and foundation for retaining wall footing	1,500,000	CY	$7.98	$12,000,000	$7,500,000
New spillway channel, common exc.	1,935,408	CY	$8.00	$15,000,000	$9,700,000
New spillway channel, rock exc.	6,333,477	CY	$32.00	$200,000,000	$130,000,000
Embankment					
Replace embankment and foundation excavated for retaining wall footing	1,500,000	CY	$1.65	$2,500,000	$1,500,000
New core	629,962	CY	$1.36	$860,000	$540,000
New filter gravel, including excavation and processing from sediment plain	178,784	CY	$4.79	$860,000	$540,000
New compacted sediment "shells," including excavation from sediment plain	7,791,962	CY	$3.81	$30,000,000	$19,000,000
Roller-compacted concrete (RCC)	1,488,529	CY	$70.20	$100,000,000	$65,000,000
Mass concrete	367,722	CY	$185.94	$68,000,000	$43,000,000
Structural concrete	539,502	CY	$288.03	$160,000,000	$97,000,000
Total				$610,000,000	$380,000,000

* Quantities and costs were estimated for the 160 ft raise. 100 ft raise option would include same features; all costs except cofferdams scaled by height of raise.

Modeling Results

A preliminary analysis of the performance of a raised SRS is presented in Appendix C. The analysis uses the sediment budget and a trapping efficiency for the raised SRS similar to that of the existing SRS when all flow passed through the outlet works. In terms of material in the range of 0.125 to 2 mm, which is linked to depositional problems in the lower Cowlitz River, the raised SRS measure decreases the cumulative sediment load at the mouth of the Toutle River by approximately 50%.

The potential performance of raising the SRS is well known as it could be very similar to the operation of the existing SRS when all flow passed through the outlet works. During this time, sedimentation in the Cowlitz River was not a problem. Further studies will explore different designs and operating procedures for the raised SRS outlet works. It may be that the Cowlitz River can handle a larger sediment load from the Toutle River without sedimentation problems, in which case the raised SRS outlet works could be designed/operated to pass more sediment, thus reducing the storage volume and cost required for a raised SRS.

Environmental Considerations

Operation of the existing fish collection facility (FCF) downstream of the SRS could improve, as the current sediment load into the facility is making operation difficult. The raised outlet works and spillway would be designed for safe downstream fish passage. There would be a negative impact to the river and tributaries upstream of the raised structure in the sediment-inundated footprint and the area would not recover within the life of the project. Figures B-10 to B-12 in Appendix B show the

areas above the SRS that would be impacted by sediment deposition for new spillway elevations of 1,050 feet, 1,100 feet, and 1,150 feet, corresponding to raises of 110 feet, 160 feet, and 210 feet.

Real Estate Considerations

As shown in Figures B-10 to B-12 in Appendix B, the areas above the existing SRS that would be impacted by sediment deposition from a raised SRS are greater than the area impacted by the existing SRS. As a result, new real estate would need to be obtained if the SRS is raised.

Discussion

The existing SRS proved successful in trapping sediment, limiting sediment deposition in the Cowlitz River, and maintaining the authorized levels of protection. Since the SRS has become run-of-river, more sediment is passing and levels of protection are decreasing. Raising the SRS would be a reliable method of managing sediment in terms of flood risk reduction on the Cowlitz River. The raised SRS would be most effective while operating with all flow passing through the outlet works, before the project again becomes run-of-river. Raising the SRS would provide a large sediment storage capacity. After implementation of the raise, and after possible short-term dredging in the Cowlitz River to remove any excess sediment coming in from the Toutle River below the SRS, no further major action would likely be required. While the construction costs presented in this Progress Report are high, the cost estimate will likely decrease as the design is refined and optimized.

The raised SRS would be designed to accommodate downstream fish passage, but not upstream volitional fish passage. For upstream fish passage, the existing FCF would be used. The FCF has been deteriorating over the years with little maintenance and repair. Since the SRS has become run-of-river, the sediment load in the river has caused major operational difficulties. If the SRS is raised, then the sediment load downstream of the SRS would be reduced, improving the operation of the FCF. Upstream of the raised SRS, the footprint of sediment deposition would be greater than the footprint for the existing SRS. This would result in more of the tributaries and habitat at the lower elevations becoming buried.

While the risk of a raised SRS would not be as great as that of a new sediment dam at Elk Rock, the increased height of a raised SRS would pose an increased risk. If the raised SRS were to fail due to a mudflow or earthquake, then a potentially larger volume of sediment would be easily erodible after the failure.

9.5.4. Measure 8 – LT-1 Sump

Background

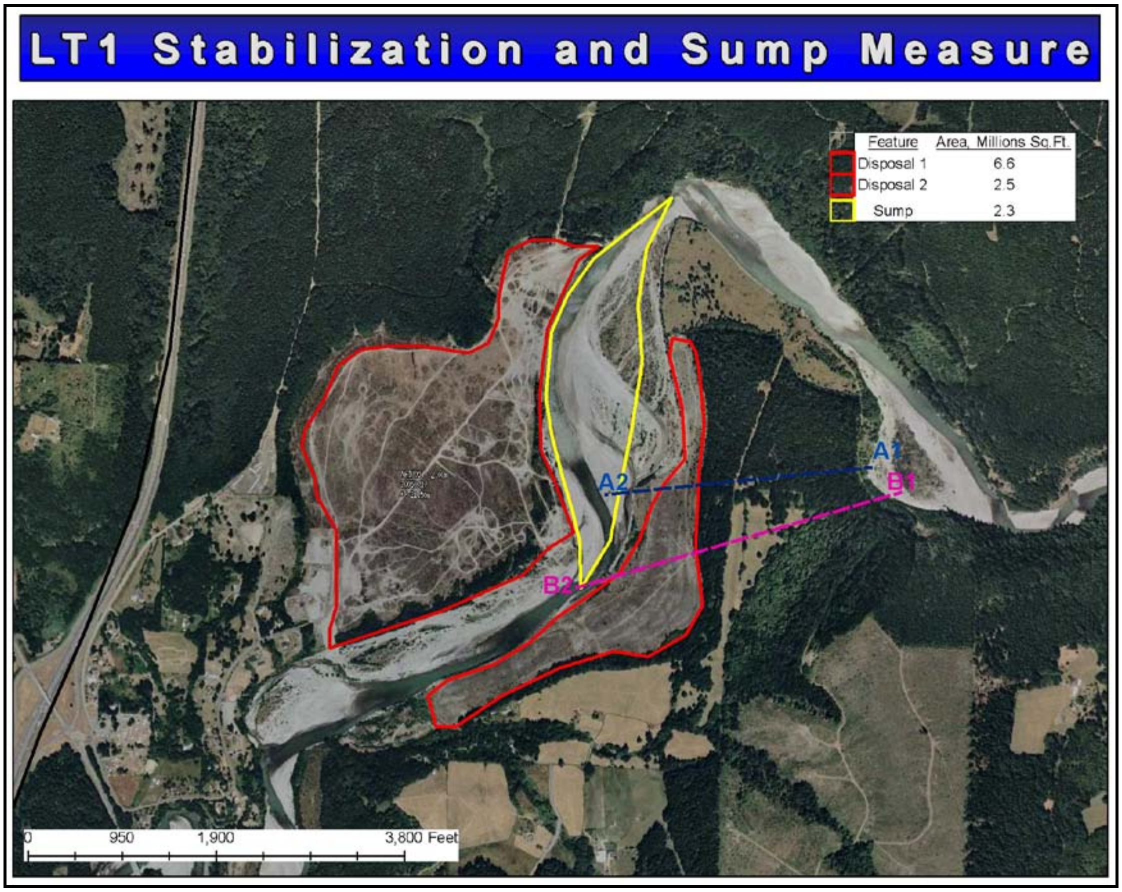
LT-1 is on the Toutle River, 1.5 river miles above the confluence with the Cowlitz River. Eight sediment basins in the Cowlitz river drainage, including LT-1, were operated from December 1980 to May 1981. Approximately 7.5 mcy of sediment was removed from the river course in this initial period. LT-1 was re-opened during the winter of 1982-1983, and an additional 3 mcy was removed from the river. LT-1 was again operated during the winter of 1983-1984 with an estimated 4.5 mcy removed. The majority of the material excavated from LT-1 was stockpiled on the right bank. The remainder was stockpiled on the left bank. The material was continuously excavated from the river bar during the contract periods (usually winter). The contractor developed a system of berms and levees that allowed control over the location of the active channel. The river was diverted back and

forth, usually daily, in order to access newly deposited material. The river was carrying a much higher sediment load during this time period than it is today.

General Description of Strategy

The plan for the measure is to operate the LT-1 site as a sump again. The 1983 Comprehensive Plan estimated 1.33 mcy could be removed from LT-1 per year. The current plan is to create a sump with a volume from 1 to 2 mcy and clean the sump annually. The removed sediment would be placed on the county-owned land on each side of the sump. Figure 22 shows the sump and disposal areas. The right bank adjacent the sump would be stabilized. Excavated material from the 1980s was placed on this bank and is currently being eroded by the river (lines A1-A2 and B1-B2 in Figure 22 represent new channel excavations considered for river diversion during sump excavation; these excavations are no longer being considered).

Figure 22. LT-1 Sump



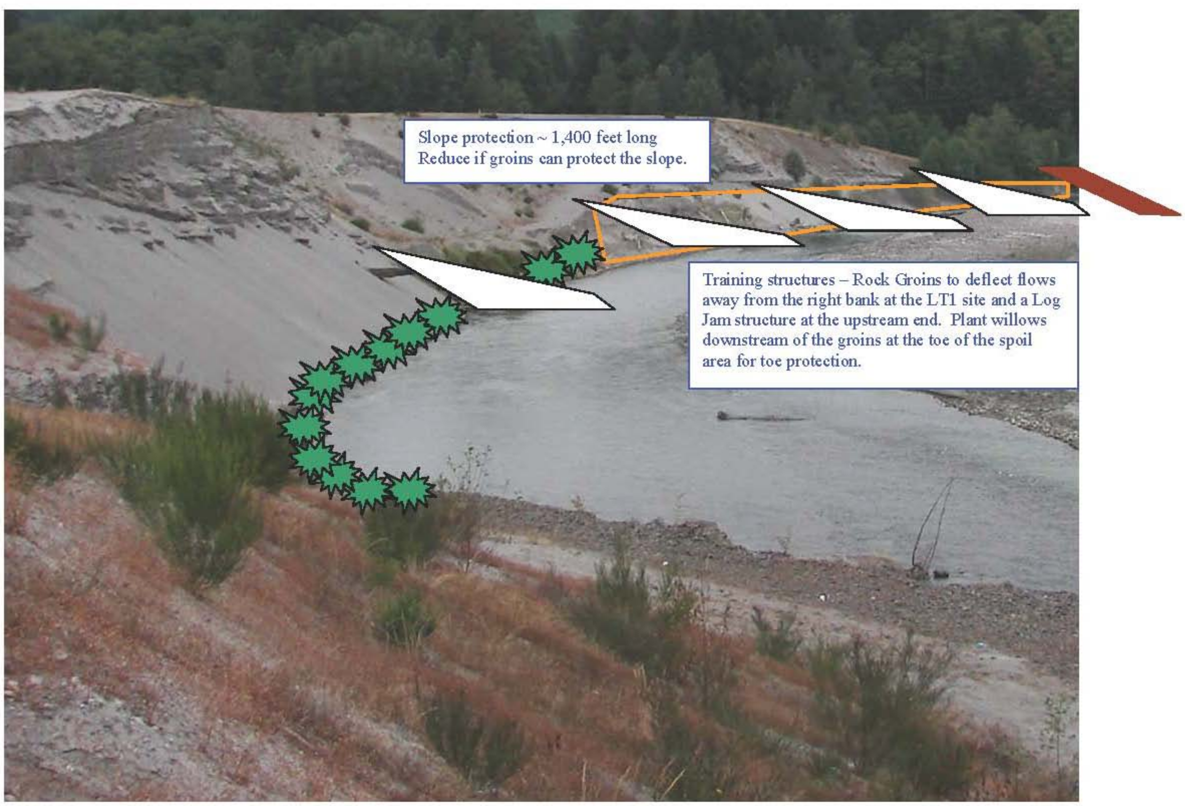
Implementation Approach

The sump would be operated as follows. The in-water work period for the LT-1 location is July through September. In year 1, a channel would be excavated along the left side of the sump area, with the removed sediment stockpiled in the left disposal area. At the beginning of July, the river would be diverted into the new channel. The sump area in Figure 22 would then be out-of-water and

the sump would be excavated, with the removed sediment stockpiled in the right disposal area. The bank stabilization measures described below would also be installed along the out-of-water right bank. Sump excavation would end by the end of September. In year 2, the river would be diverted to the right side of the sump area, with the removed sediment stockpiled in the right disposal area. The sump area would then be out-of-water again and would be excavated, with the removed sediment stockpiled in the left disposal area. The procedure would repeat annually. It is anticipated that excavation would occur using either scrapers or excavators and trucks depending on the groundwater elevation. To allow for scraper or excavator/truck access, a haul road from the sump to each disposal site would be constructed and removed afterward, if required. Estimated disposal area capacities are 14.5 mcy for the right bank area and 5.5 mcy for the left bank area, for a total of 20 mcy.

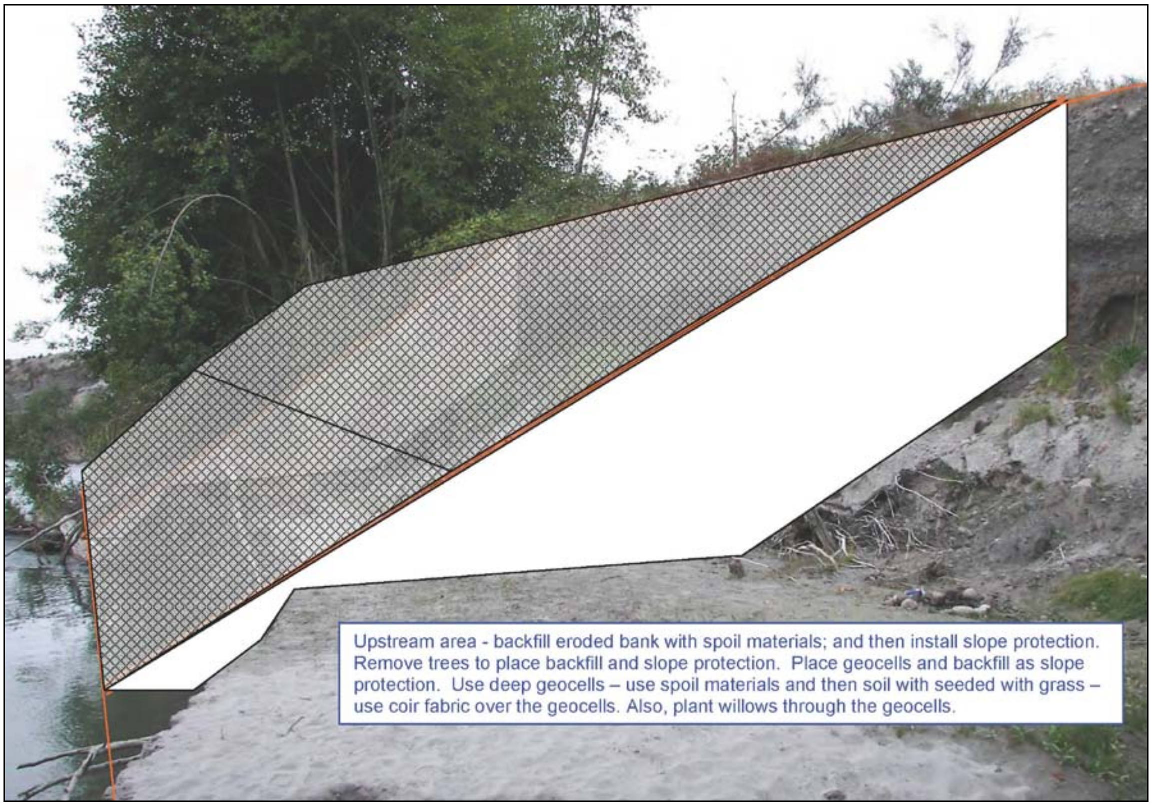
Alternatives were considered for the right bank protection methods. These included rock dikes or groins, geotubes, rip rap, concrete articulated mattresses, geocells, and log jams. The proposed alternative is shown in the four photos in Figure 23. It includes a log jam at the upstream end, rock groin structures with large woody debris to direct high flows away from the bank, geocell bank protection between the groins, and willow plantings.

Figure 23. LT-1 Sump Proposed Alternative Photos



General Layout of LT1 streambank protection features

Figure 23 (continued). LT-1 Sump Proposed Alternative Photos



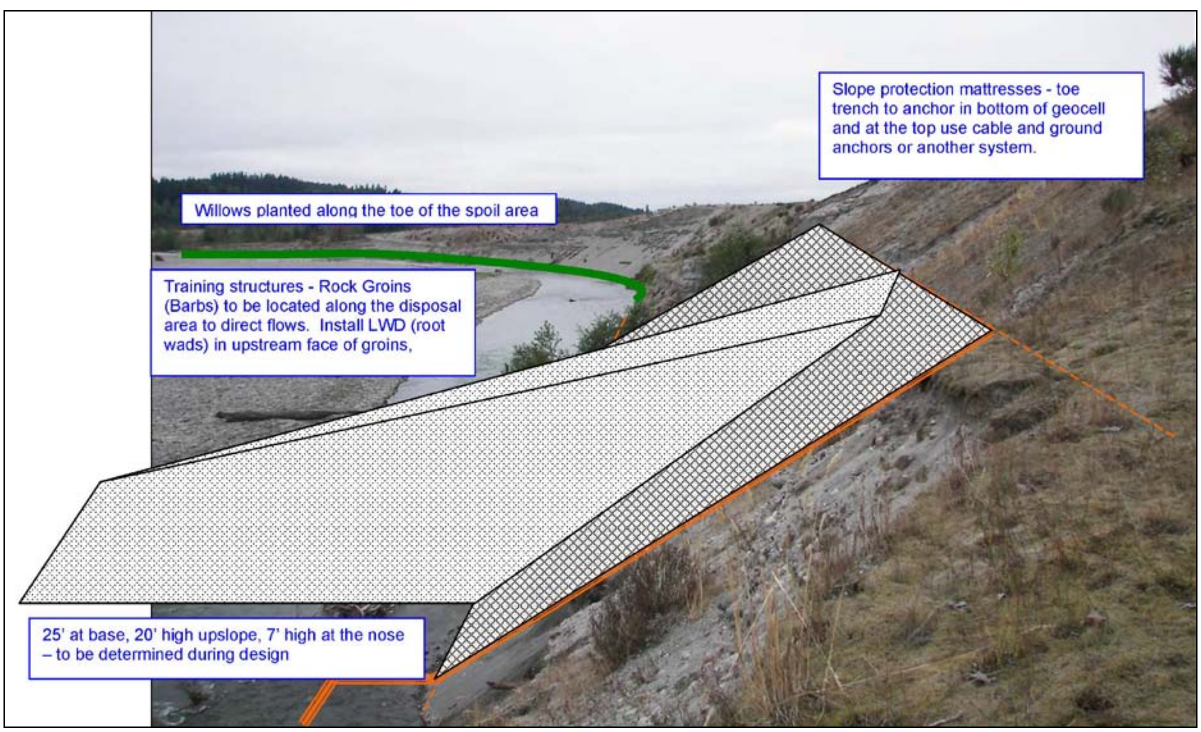
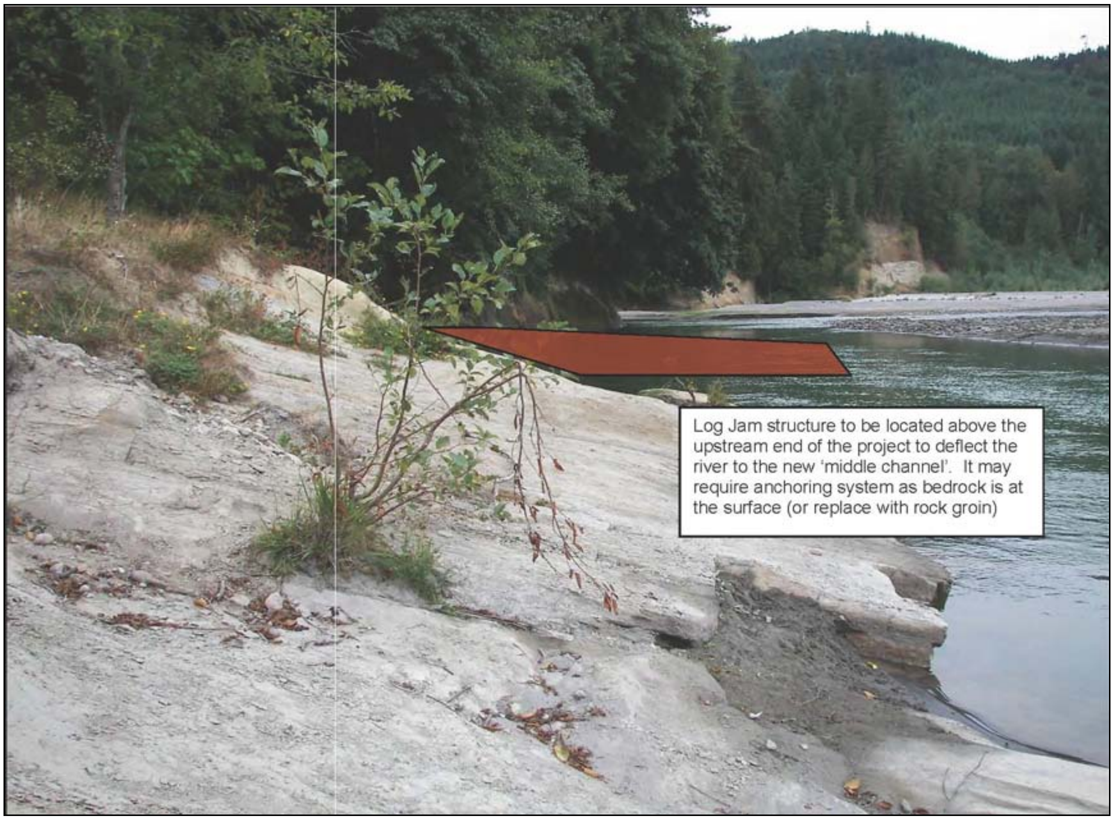


Figure 23 (continued). LT-1 Sump Proposed Alternative Photos



Modeling Results

A preliminary analysis of the performance of a sump at LT-1 is presented in Appendix C. The analysis includes the LT-1 sump in combination with a raised SRS and in combination with a sediment plain grade-control concept. The results of the combination with the grade-control concept are not presented here as there is still much uncertainty in the potential implementation and performance of the grade-building structures concept. The analysis uses the sediment budget and a trapping efficiency based on the geometry of the proposed LT-1 sump. In terms of material in the range of 0.125 to 2 millimeters, which is linked to depositional problems in the lower Cowlitz River:

- The raised SRS decreases the cumulative sediment load at the mouth of the Toutle River by 47% to 53%.
- The raised SRS plus LT-1 sump decreases the cumulative sediment load at the mouth of the Toutle River by 51% to 58%.

The impact of the LT-1 sump is only a sediment load reduction of 4% to 5%. The sump is only effective at trapping bedload material. All sediment in suspension passes through the sump.

Stabilization of the dredge disposal site on the right bank may be worthwhile. By comparing aerial photos, the estimated erosion volume from 1999 to 2006 was 200,000 cy or approximately 28,800 cy per year on average. The portion of the bank that is medium sand and coarser, and some of the fine sand, is likely depositing in the Cowlitz River. It may turn out that the LT-1 bank source is a

significant enough source of sediment that deposits in the Cowlitz River that it is worthwhile from a cost point of view to stabilize the bank.

Cost Estimate

The estimated annual mobilization/demobilization cost is $500,000. The estimated cost per cubic yard for excavating and disposing of sediment is $5/cy. Based on an assumed average annual sediment removal volume of 1.33 mcy, the annual sump cost is $6.65 million. The estimated annual cost for haul roads is $400,000. The estimated one-time cost for stabilizing the right bank is $2 million. This results in a year 1 cost of $10 million and subsequent annual costs of $8 million.

Environmental Considerations

The in-water work period in the Toutle River is July through September. Working within this time period to divert the river to the side of the area in order to remove sediment from the sump in the dry should reduce impacts to fish. Fish stranding concerns have been raised with a sump in the river. The concerns involve the formation of pools disconnected from the river as a result of sedimentation in the sump, and the potential for fish stranding in the pools.

Real Estate Considerations

Expected operation of the LT-1 sump and its disposal areas are within current real estate county boundaries covered under the LT-1 sump real estate agreements.

Discussion

The use of LT-1 as a sump is not considered an effective approach if sediment in suspension at the LT-1 site is not depositing in large quantities in the Cowlitz River. If long-term analysis of deposition indicates that bedload at LT-1 is depositing in the Cowlitz River in sufficient quantities to affect the level of protection in the communities on the river, then reactivation of LT-1 may be an appropriate and cost effective measure. However, it may be worthwhile to stabilize the dredge disposal site on the right bank because a large percentage of this sediment source is of a grain size that deposits in the Cowlitz River.

9.5.5. Measure 10 – Modified Operation of Mossyrock Dam

General Description of Strategy

This measure proposes to use flows from Mossyrock Dam to either scour out sediment from the lower Cowlitz and/or increase the sediment transport capability of the Cowlitz during flood events to reduce the amount of deposition in the lower Cowlitz River. Two general approaches were investigated:

1. Drawdown Flushing. Re-regulation of fall drawdown to winter flood control storage whereby water is evacuated from the pool prior to flood season with a higher pulse, causing scour of sediment in the lower Cowlitz River.
2. Rain Event Flushing. Rain event re-regulation whereby water is released at a higher rate immediately after a large rain event, reducing the amount of deposition in the lower Cowlitz from sediment input from the Toutle River.

Implementation Approach

The Cowlitz River at Castle Rock has been regulated by Mossyrock Dam (Riffe Lake) and Mayfield Dam (Mayfield Lake) since water year 1969 (Figure 24). These two reservoirs are part of the Cowlitz Project which is owned and operated by the City of Tacoma, Washington (Tacoma Power Company). Riffe Lake provides 360,000 acre-feet of flood control storage during December and January. Mayfield Lake acts as a re-regulating reservoir for releases from Mossyrock Dam. During the peak of the flood season (December and January), 360,000 acre-feet of flood control storage is available with the downstream flow objective of keeping the flow below 70,000 cfs at Castle Rock. A second maximum release objective limits releases below Mayfield Dam to 25,000 cfs to prevent flooding in communities along the Cowlitz between Mayfield Dam and Castle Rock.

Two general flushing concepts were investigated as described below:

1. <u>Drawdown flushing.</u> Re-regulation of fall drawdown to winter flood control storage whereby water is evacuated from the pool prior to flood season with a higher pulse.
2. <u>Rain event flushing.</u> Rain event re-regulation whereby water is released at a higher rate immediately after a large rain event.

For each general flushing concept, two re-regulation hydrographs were developed:

1. <u>25,000 cfs max release</u>. This scheme releases a maximum of 25,000 cfs from Mayfield Dam while not exceeding a maximum flow at Castle Rock of 50,000 cfs.
2. <u>70,000 cfs control</u>. This scheme regulates below a maximum flow of 70,000 cfs at the Castle Rock gage and allows for releases from Mayfield in excess of 25,000 cfs. This scheme is not feasible without development of additional flood protection projects on the Cowlitz, but is informative concerning sensitivity of deposition related to regulated flows.

Modeling Results

An existing uncalibrated mobile-bed HEC-RAS model of the lower Cowlitz River was run for water years 2007 and 2008 with existing condition hydrology and four re-regulation inputs. The model runs were used to investigate the relative change in deposition in the Lower Cowlitz due to the flushing schemes. Figure 25 shows the model geometry and boundary condition inputs required for mobile bed HEC-RAS. The only input modified for the flushing flow runs was the Cowlitz River inflow. The four re-regulation inputs analyzed were:

1. Drawdown flushing with 2,000 cfs max release.
2. Drawdown flushing with 70,000 cfs control at Castle Rock.
3. Rain event flushing with 25,000 cfs max release.
4. Rain event flushing with 70,000 cfs control at Castle Rock.

Appendix C includes detailed analyses of these four schemes. Re-regulation of flood protection projects on the Cowlitz can result in decreased deposition in the lower Cowlitz. Existing maximum release limitations in place due to flooding on the Cowlitz between Mayfield Dam and Castle Rock reduce the potential for flushing considerably. With current limitations, the drawdown pulse results in a marginal decrease in deposition. A greater potential for moving sediment lies in re-regulation of large storm events in the upper Cowlitz. Model results indicate that deposition in the lower Cowlitz could be reduced by as much as 12% on a biannual basis if a flow release from the regulation projects is triggered by a sizeable storm on the Toutle.

Figure 24. Cowlitz Watershed and Regulation Projects

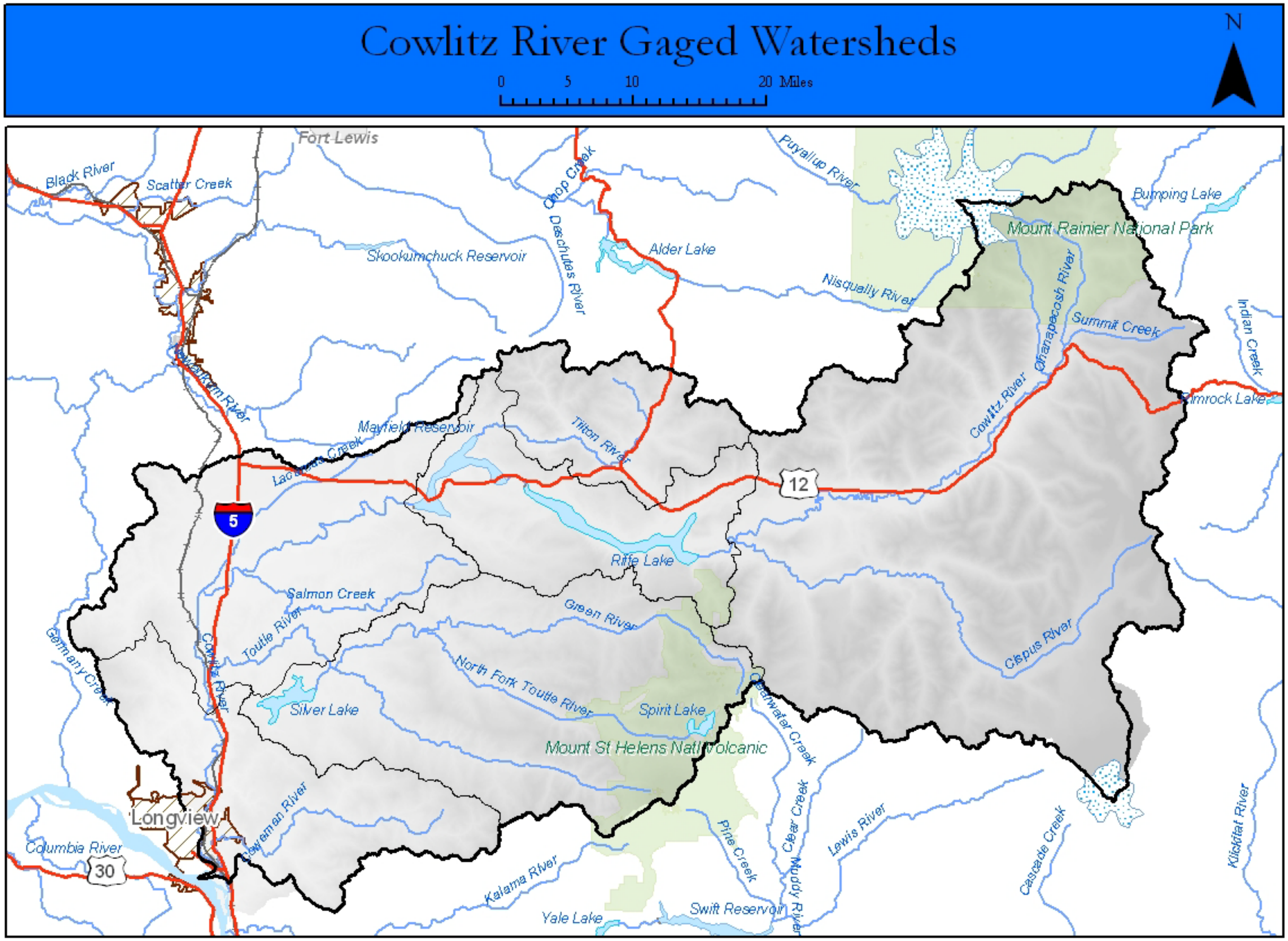
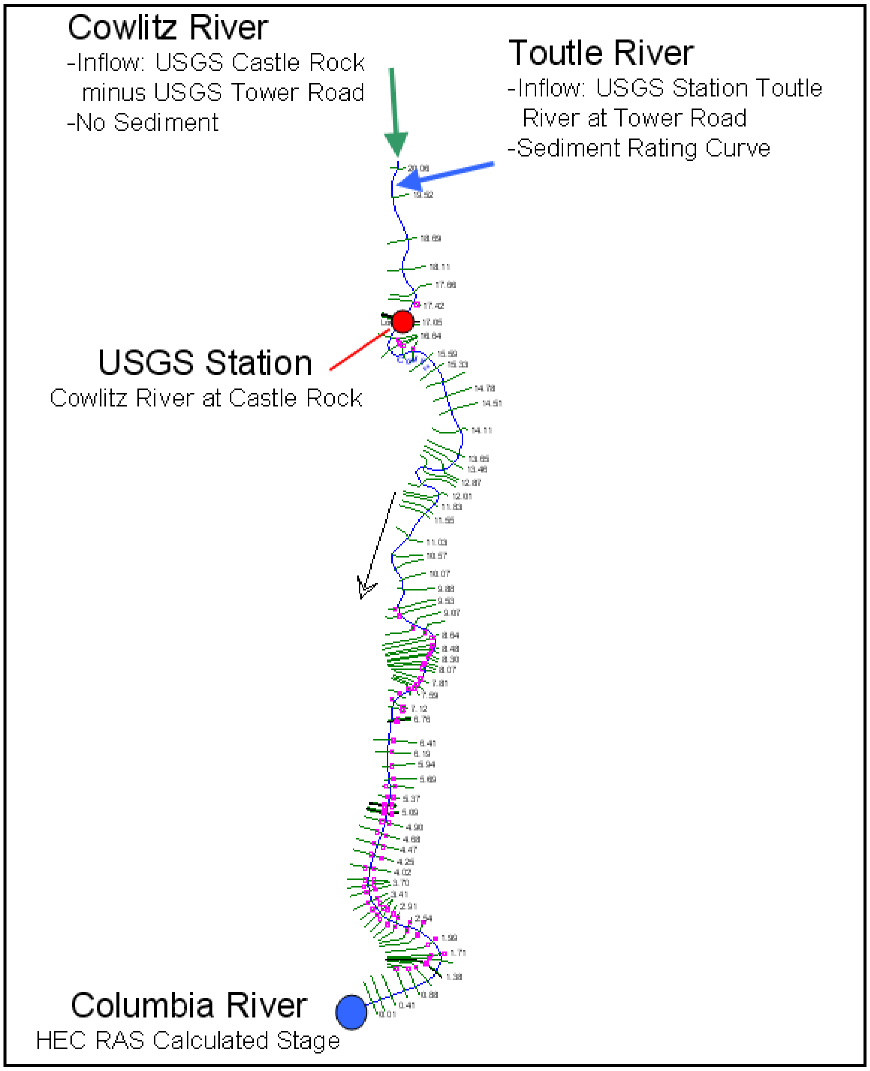


Figure 25. Mobile Bed HEC-RAS Geometry and Inputs for Flushing Flow Analyses



Cost Estimate

At the time of this Progress Report, a cost estimate has not been generated. The Corps is only early in the process of discussing possibilities with the owner of Mossyrock Dam, Tacoma Power Company. The largest cost associated with this measure will involve economic impacts to the power company due to lost power generation.

Environmental Considerations

Adverse environmental impacts are associated with the fall drawdown flushing scenarios. Flow is usually low in the Cowlitz River in the September time period when drawdown flushing would occur. High flushing flows during this time period would not match natural conditions, and the high flows could scour salmon eggs and cause turbidity concerns. The rain event flushing scenarios would not pose environmental concerns because flows in the Cowlitz River would already be high.

Real Estate Considerations

No real estate would need to be acquired for this measure.

Discussion

The most promising use of flushing flows appears to be the re-regulation of Mossyrock Dam during rain events. The amount of sediment depositing in the lower Cowlitz from sediment-laden Toutle River flows can be decreased by releasing high flows of sediment-free water from Mossyrock Dam to reduce the amount of deposition as Toutle River flow recedes. In order to take advantage of this approach, close coordination would be required between the Corps and Tacoma Power Company during rain events. Agreements would have to be in place to regulate these events and to reimburse the power company for lost power generation revenue. In addition, more study would be required to identify potential adverse impacts involved with the re-regulation.

9.5.6. Measure 12 – Cowlitz River Dredging

General Description of Strategy

This measure involves dredging in the Cowlitz River to remove sediment that is reducing the conveyance of the river and increasing flood risk. Due to uncertainties in the Cowlitz River sediment budget, the amount of future deposition throughout the Cowlitz remains speculative. However, using the sediment budget information allows evaluation of various options. To remove material from problematic locations along the Cowlitz River, two methods were considered: dredge pipelines to accommodate larger volumes, and dragline dredging in the case of lesser deposition volumes.

Implementation Approach

General Information. Figure 26 shows the average river grade of the lower Cowlitz as 0.03% over a span of 17 miles. Pre-determined river reaches will be dredged annually for 30 days (the in-water work period is the month of August) to remove deposited fine to coarse sized sand. For planning and cost estimation purposes, Figure 27 shows four reaches that have been identified based on disposal site locations, dredging access, and forecasted deposition volumes.

Figure 26. Cowlitz River Grade-lines Created from 2007 LiDAR

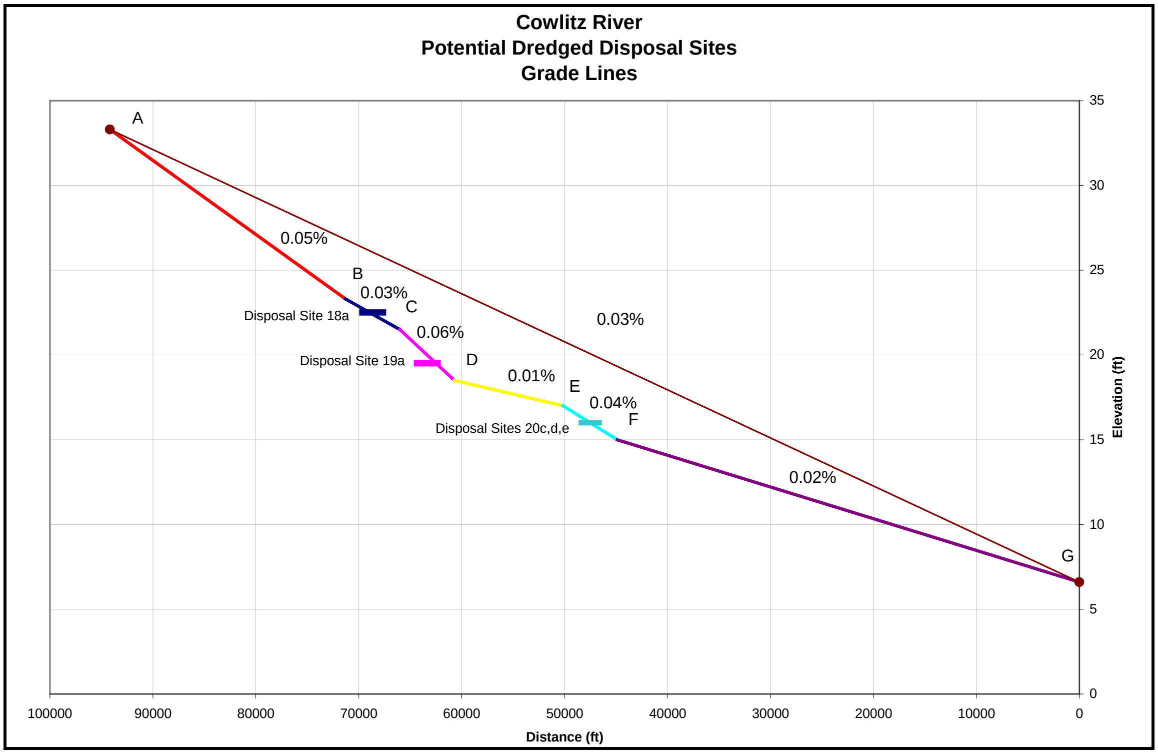
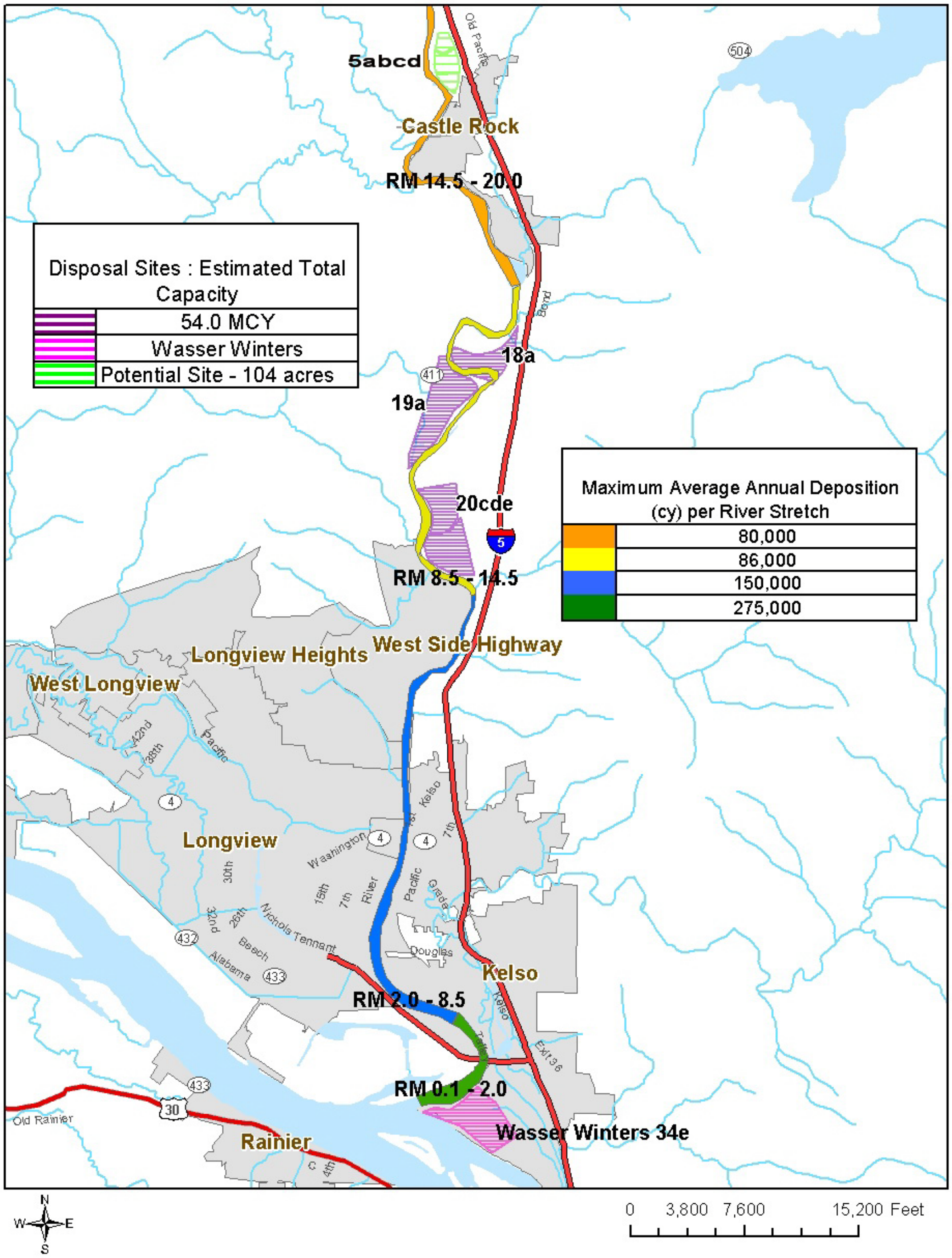
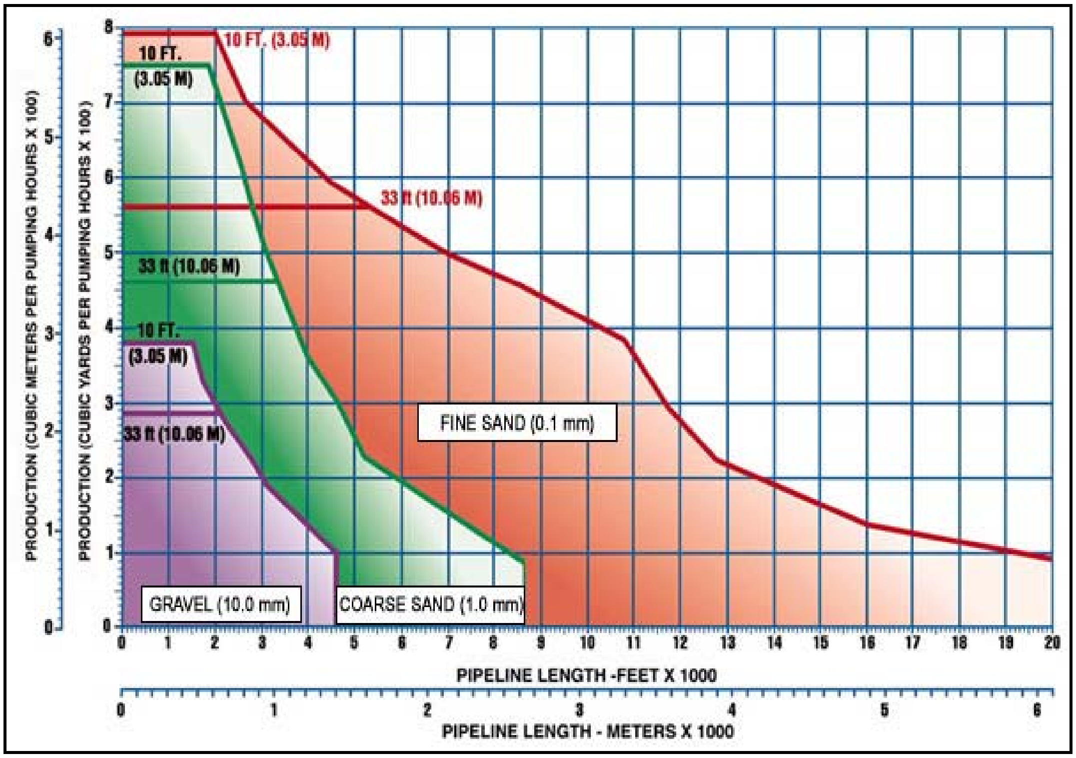


Figure 27. Dredging/Disposal Areas and Estimated Maximum Average Annual Deposition



Pipeline Specific. In order to remove the larger volumes of material, 12-inch pipeline dredges would be used for the following river reaches. Pipeline pumping distances in relation to material size are shown in Figure 28.

Figure 28. Pipeline Pumping Distances in Relation to Material Size



From RM 0.1 to 2.0, one dredge would direct pump to the Wasser Winters disposal site, located along the southern bank of the Cowlitz River mouth. The average annual maximum deposition for RM 0.1 to 2.0 is approximately 275,000 cy. With a production rate of 8,000 cy per day, it would take 34 days to remove. The average annual minimum deposition is 52,000 cy. With a production rate of 8,000 cy per day, it would take 7 days to remove.

From RM 2.0-8.5, one dredge would pipeline pump either upstream to disposal site 20cde or downstream to the Wasser Winters site. Pumping distances would not exceed 6.0 miles. The longer pumping distance would require four boosters spaced along the system. Two dredges could be used to minimize pumping distances and number of boosters necessary per system. The average annual maximum deposition for RM 2.0 to 8.5 is approximately 150,000 cy. With a production rate of 5,500 cy per day, it would take 27 days to remove. The average annual minimum deposition is 25,000 cy. With a production rate of 5,500 cy per day, it would take 5 days to remove.

From RM 8.5 to 14.5, one dredge would direct pump to disposal sites 20cde, 19a, and 18a, located along both banks of the Cowlitz River. The average annual maximum deposition for RM 8.5-14.5 is approximately 86,000 cy. With a production rate of 8,000 cy per day, it would take 11 days to remove. The average annual minimum deposition is 13,000 cy. With a production rate of 8,000 cy per day, it would take 2 days to remove.

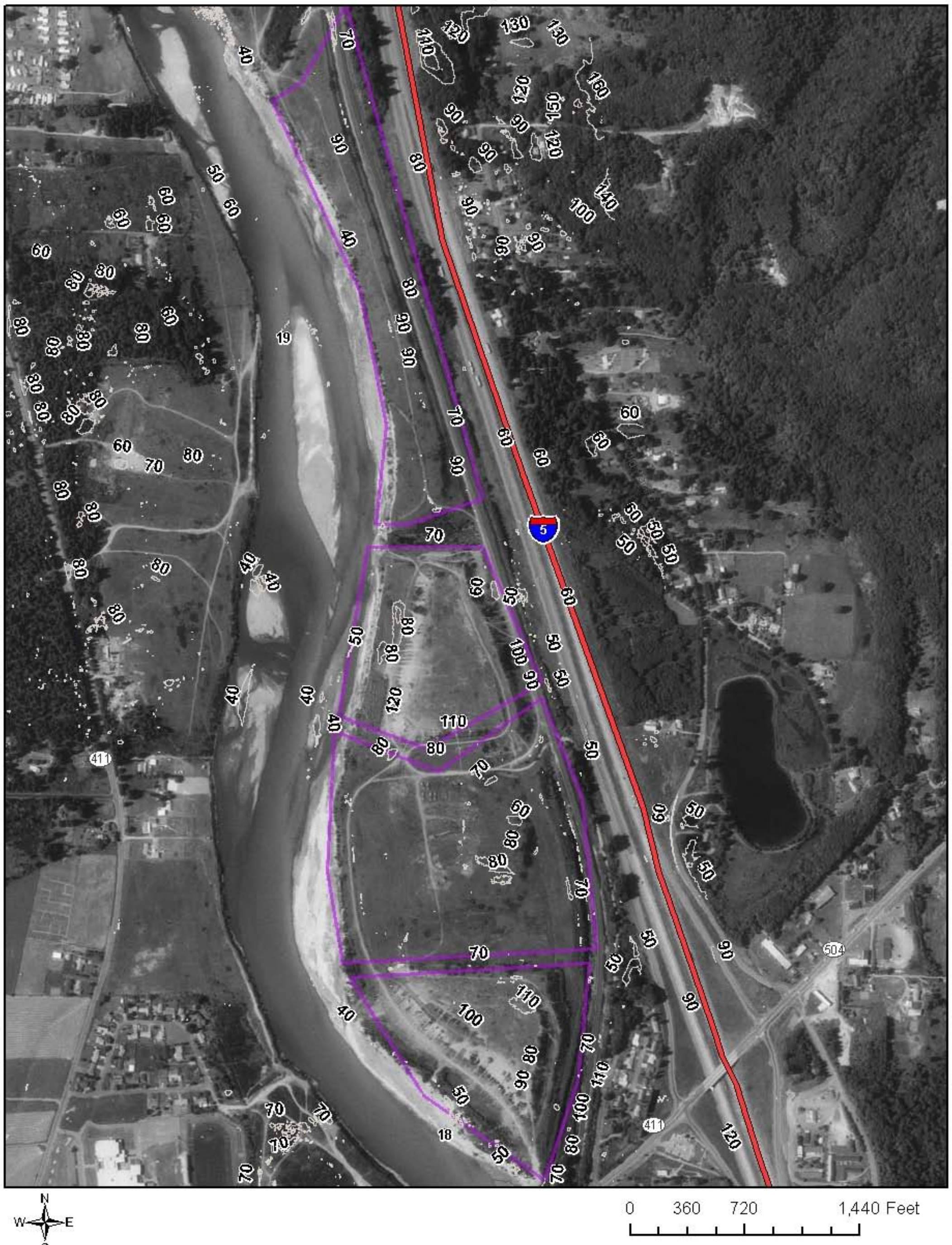
From RM 14.5-19.5, one dredge would direct pump to disposal site 5abcd or pipeline pump downstream to disposal site 18a. Pumping distances would not exceed 3.0 miles. The longer pumping distances would require two boosters spaced along the system. Two dredges could be used to minimize pumping distances and number of boosters necessary. The average annual maximum deposition for RM 14.5-19.5 is approximately 80,000 cy. With a production rate of 5,500 cy per day, it would take 15 days to remove. The average annual minimum deposition is 3,000 cy. With a production rate of 5,500 cy per day, it would take 1 day to remove.

Dragline Specific. The dragline method of dredging was considered in case lower volumes of material will deposit annually. It is assumed that a single dragline will be able to remove about 350 cy per day. If necessary, multiple draglines could be deployed within a given dredge reach simultaneously. For the sediment volumes shown in Figure 27 (maximum average annual), dragline dredging did not prove to save costs over pipeline dredging. Difficulties with dragline dredging include the high number of draglines needed to remove the sediment within the short in-water work window and attaining river access for the draglines to operate. For these reasons dragline dredging is not evaluated to the same level of detail as pipeline dredging. If conditions change, dragline dredging may be reconsidered; for example, the sediment volume to remove changes to a lower volume. The Castle Rock area in particular may be suitable for dragline dredging.

Disposal Sites. Four potential disposal areas have been identified along the Cowlitz River from RM 9-19. Figure 27 shows the location of these sites in relation to the proposed dredging reaches and impacted communities. Lifts from river bottom to disposal sites range from 20-35 feet. It is important to note that no contact with landowners has been made to determine land availability. The assessed property value information is used for planning purposes only. If the future decision is to proceed with further assessment, discussions with landowners would occur.

Disposal site 5abcd is located between RM 18-19 and is about 135 acres in size with a dredge material storage capacity of 2.9 mcy. The assessed property value is $400,000. Figure 29 shows a plan view of the site with elevations. A barrier may need to be constructed along the east edge of the site to protect outlying areas. This can be accomplished by using dredge material as the site is filled.

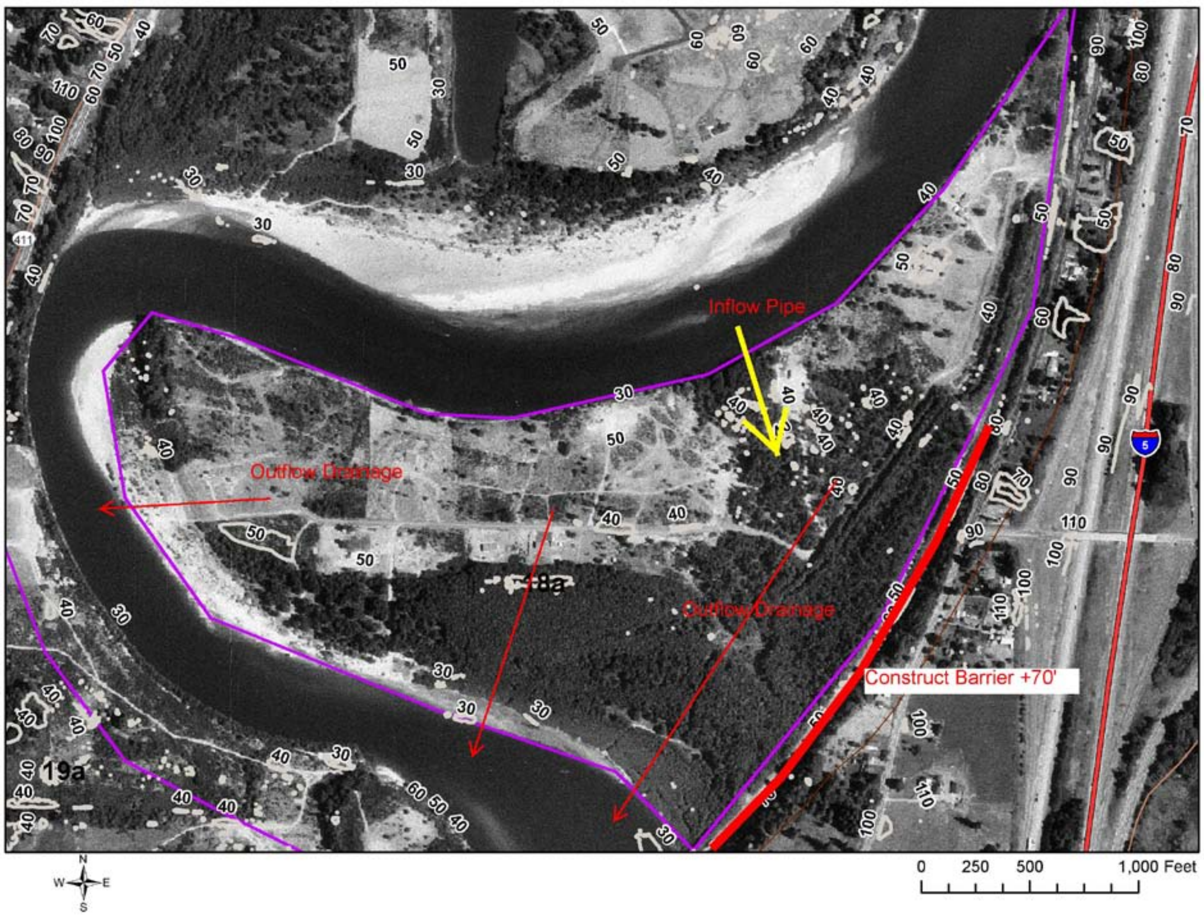
Figure 29. Disposal Site 5abcd



Horseshoe Bend (labeled as site 18a) is located between RM 12-14. It is about 165 acres in size with a dredge material storage capacity of 15 mcy. The assessed property value is $3 million. Figure 30 shows a plan view of the site with elevations. A barrier may need to be constructed along the west edge of the site to protect outlying areas. This can be accomplished by using dredge material as the site is filled.

Figure 30. Disposal Site 18a (Horseshoe Bend)

The lowest bank line elevations along Horseshoe Bend exist on the left bank upstream of the meander. Outflow drainage will exit downstream at the far end of the disposal area.



Disposal site 19a is located between RM 10.5-12.5. It is about 208 acres in size, with a storage capacity of 19 mcy of dredge material. The assessed property value is $3 million. Figure 31 shows a plan view of the site with elevations. A barrier may need to be constructed along the east edge of the site to protect outlying areas. This can be accomplished by using dredge material as the site is filled.

Disposal site 20cde is located between RM 8.5-10.5. It is about 261 acres in size, with a storage capacity of 20 mcy of dredge material. The assessed property value is $3 million. Figure 32 shows a plan view of the site with elevations. A barrier may need to be constructed along the west side of the site to protect outlying areas. This can be accomplished by using dredge material as the site is filled.

Figure 31. Disposal Site 19a

The easiest pipeline access point to site 19a exists on the right bank upstream of the meander. Outflow drainage will exit downstream at the far end of the disposal area.

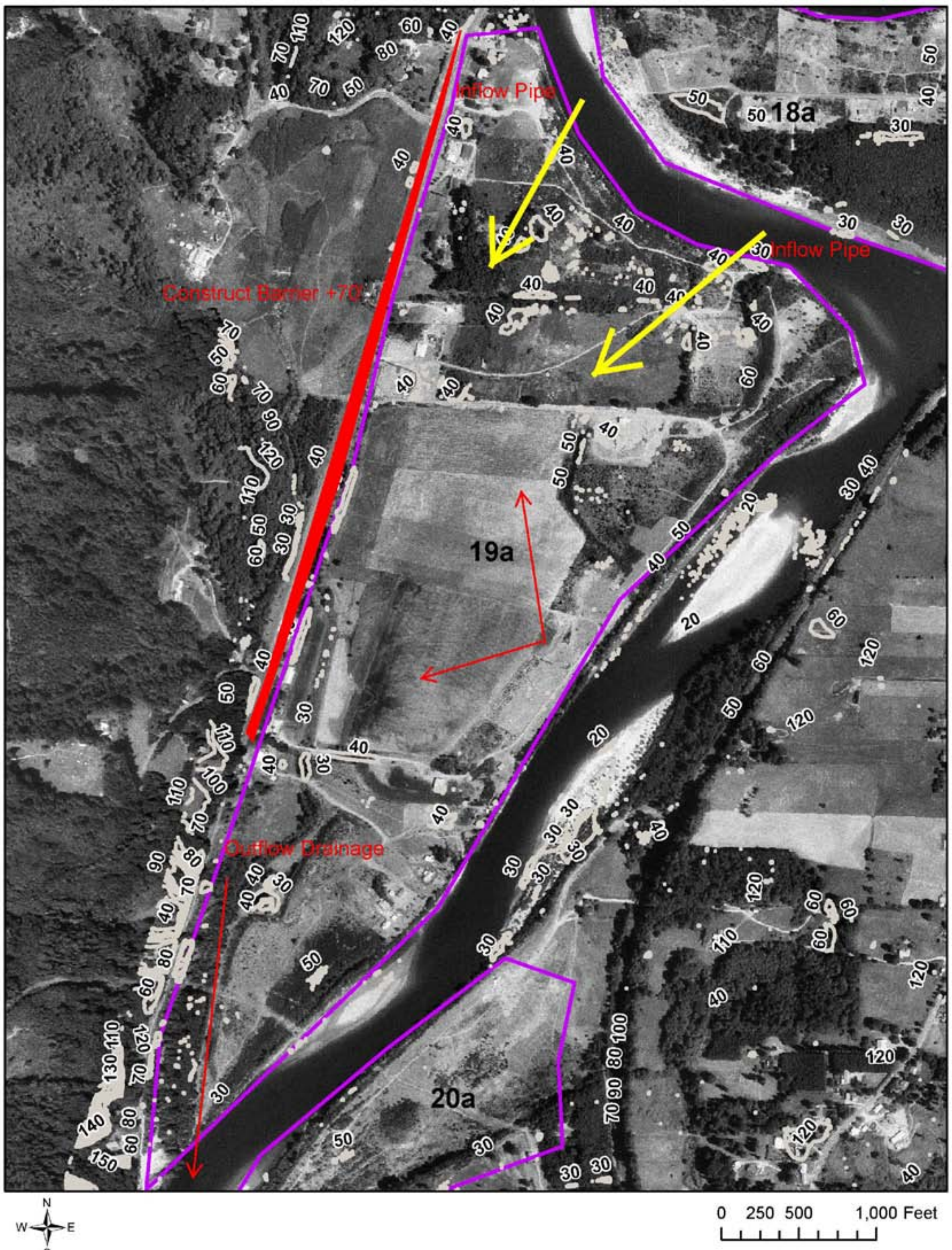
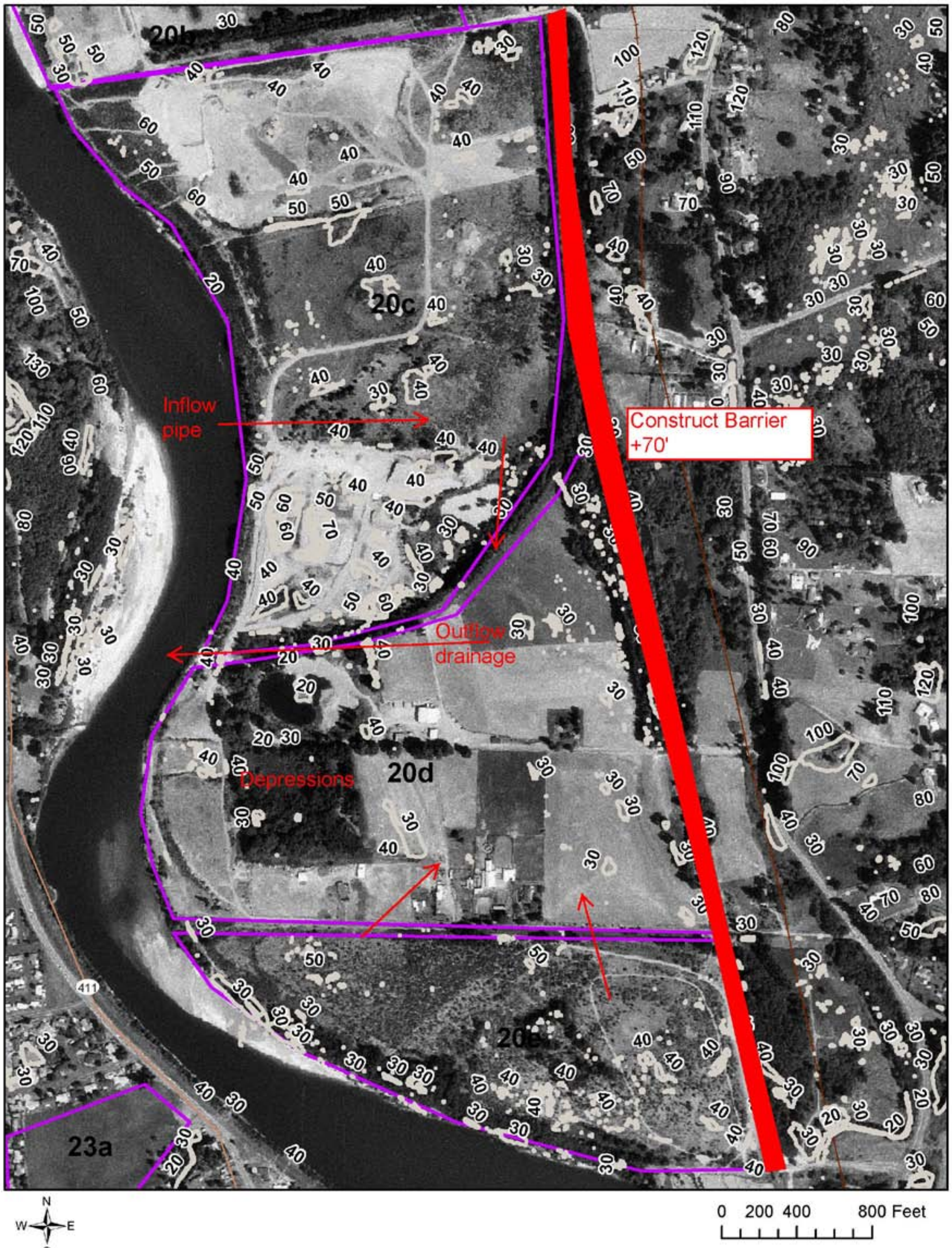


Figure 32. Disposal Site 20cde

The easiest pipeline access point to site 20cde exists on the left bank upstream of the meander. Outflow drainage will exit downstream towards the middle of the disposal area.



Cost Estimate

Figure 33 shows a cost estimate for pipeline dredging for each river reach. The unit price per cubic yard for removal on river reach 0.1 to 2.0 is $6.29. Unit price per cubic yard for removal on river reach 2.0 to 8.5 is $46.22. Unit price per cubic yard for removal on river reach 8.5 to 14.5 is $9.39. Unit price per cubic yard for removal on river reach 14.5 to 19.5 is $42.80.

For disposal sites, demolition and removal would need to occur prior to site prep and adds an extra cost. The total assessed property value for the three sites is $9.5 million. Site preparation estimates referenced the Wasser Winters upland preparation estimates and were based on the relationship between acreage and effort. Total site preparation estimates range from $3.2 million to $4.2 million.

The total costs for Cowlitz River dredging are summarized in Table 6. Costs are based on minimum (93,000 cy) and maximum (591,000 cy) expected annual deposition totals. **The mean expected annual deposition total is 240,000 cy. For this volume, the annual dredging cost would be approximately $6 million, and the total cost through 2035 would be approximately $164 million.**

Table 6. Total Costs for Cowlitz River Dredging

Item	Minimum Cost ($ millions)	Maximum Cost ($ millions)
Disposal site acquisition & preparation	14.0	14.0
Annual dredging cost	2.5	13.0
Total cost through 2035	76.5	339.0
Total cost through 2060	139.0	664.0

Environmental Considerations

Impacts would occur during both in-channel removal and upland disposal. Dredged areas and disposal sites would be disturbed annually leaving them unable to reestablish. As with all dredging operations, the chance of oil leaks exists. Also, dredging operations may increase turbidity.

Real Estate Considerations

All land identified in this proposal as potential disposal sites along the Cowlitz River is privately owned and would need to be purchased, prepped, and efficiently managed. The acquisition of these properties may be time extensive and costly to the project as many of the properties are currently developed. Currently, no discussions have occurred with landowners.

Discussion

When considered from the long-term standpoint, a dredging option may appear to be a costly, intrusive solution. Dredging would be an annual cost of approximately $2,500,000 to $13,000,000 occurring during August of each year. However, when coupled with other measures this measure allows for flexibility in cost, degree of environmental impact, and dealing with future sediment load uncertainties. The Cowlitz River dredging measure would allow the natural processes of erosion, sediment transport, and sediment deposition occurring within the river system to continue in an unregulated environment.

Figure 33. Dredging Cost Estimate

	Reach 1 (RM 0.1-2)			Reach 2b (RM 2-8.5)			Reach 2c : Disposal Areas (RM 8.5-14.5)			Reach 3 (RM 14.5-19.5)		
	2035	Max Expected Annual	Min Expected Annual	2035	Max Expected Annual	Min Expected Annual	2035	Max Expected Annual	Min Expected Annual	2035	Max Expected Annual	Min Expected Annual
PIPELINE COSTS												
Annual Qty Removed	6,471,735	258,869	53,021	3,586,745	143,469.79	24,951.27	2,027,290	81,092	15,595	1,871,345	74,854	3,119
Estimated Production	8,000	8,000	8,000	5,500	5,500	5,500	8,000	8,000	8,000	5,500	5,500	5,500
Days to Remove	809	32	7	652	26	5	253	10	2	340	14	1
Estimated Daily Rate	$39,000	$39,000	$39,000	$79,000	$79,000	$79,000	$39,000	$39,000	$39,000	$59,000	$59,000	$59,000
Total Dredging Cost	$31,549,708	$1,261,988	$258,480	$51,518,696	$2,060,748	$358,391	$9,883,041	$395,322	$76,023	$20,074,428	$802,977	$33,457
Estimated Mobilization*	$390,000	$390,000	$390,000	$4,790,000	$4,790,000	$4,790,000	$390,000	$390,000	$390,000	$2,590,000	$2,590,000	$2,590,000
Overall Removal Cost	**$31,939,708**	**$1,651,988**	**$648,480**	**$56,308,696**	**$6,850,748**	**$5,148,391**	**$10,273,041**	**$785,322**	**$466,023**	**$22,664,428**	**$3,392,977**	**$2,623,457**
Unit Price	$4.94	$6.38	$12.23	$15.70	$47.75	$206.34	$5.07	$9.68	$29.88	$12.11	$45.33	$841.15
Assumptions	Direct Pump			4 booster pumps			Direct Pump			2 booster pumps		
	*Single Mobilization Cost											

Note: For small dredge volumes resulting in pipeline unit costs above $50/cy, it is assumed that dragline dredging could potentially be employed to limit the removal unit cost to $50/cy.

9.5.7. Measure 13 – Expand Floodplain on Cowlitz River

General Description of Strategy

The expanded floodplain measure decreases flood stages in the lower Cowlitz River by restoring the natural floodplain terrace along portions of the lower 20 miles of the river. Levees and infrastructure are set back and dredge spoil/fill above the historic floodplain terrace are removed, increasing conveyance during flood flows and lowering flood stages. The setback and excavated area would be managed as a flood protection measure and remain as managed greenspace.

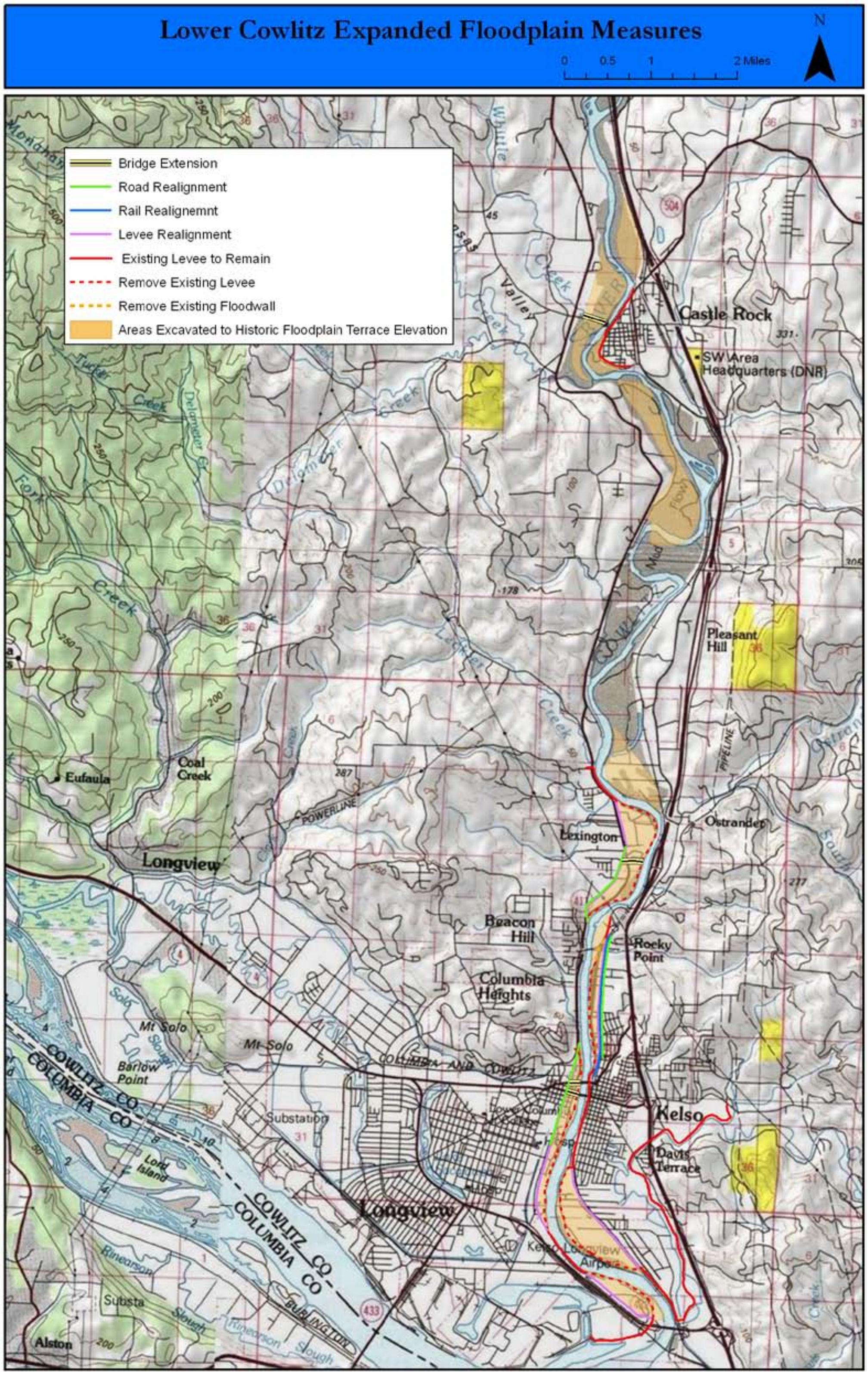
This measure is only a concept. Discussions with landowners have not occurred because no determination has been made that this option is viable.

Implementation Approach

Figure 34 shows the suite of activities that combine to make the expanded floodplain measure. The activities shown in Figure 34 represent an aggressive expansion of the floodplain to investigate the potential of the measure to reduce flood stages. The combined activities have a cumulative effect with downstream measures providing benefit for some distance upstream. The area proposed for floodplain expansion is largely privately owned with a mix of residential, commercial, industrial and agricultural uses. Floodplain expansion along the Longview and Kelso levees effects infrastructure most greatly involving relocation of levees, rail lines, roadways, as well as extension of two bridges and removal of dredge spoils in the setback area. Expansion along the Lexington levee alleviates the existing constriction at Rocky Point with a large setback of the Lexington levee, re-terracing the reclaimed floodplain and the extension of one bridge. Setback of the Castle Rock levee was not required to reduce flood stage due to the lack of geographic constraints on the opposite bank. Significant dredge spoil removal and the extension of one bridge comprise the activities in the vicinity of Castle Rock.

The setback areas would be re-terraced to inundate during events larger than the 50% to 20% annual exceedance probability (AEP) flood flows. Average annual sediment transport capacity is not expected to change with this measure as it does not modify the river below historic bank elevations. During more extreme flood events, silts and fine sands are abundantly supplied by the Toutle River and observed depositing in existing connected floodplain terraces along the lower Cowlitz River. Expansion of the floodplain would likely induce more deposition in the floodplain during these extreme events as average velocities in floodplains would be decreased. Aging floodplains would vegetate resulting in rougher overbanks and a further decrease in off-channel velocities causing additional off-channel deposition. It is expected that continued deposition in the expanded floodplain would raise the terrace and reduce the effectiveness of the measure without occasional maintenance of the created greenspace. This maintenance would include periodic removal of deposited soils, clearing of understory vegetation and thinning of trees.

Figure 34. Expanded Floodplain Measure



Modeling Results

Appendix C includes a detailed analysis of the expanded floodplain measure described above. Table 7 provides the results of the analysis for the 1% and 0.5% AEP flows, which bound the LOP flows.

Table 7. Average Reduction in Stage Due to Expanded Floodplain Measure (feet)

Site	1% AEP Flow	0.5% AEP Flow
Longview Levee	0.2	0.2
Kelso Levee	0.2	0.4
Lexington Levee	1.4	1.9
Castle Rock Levee	1.9	2.3

The measure has limited ability to reduce flood stages in the LOP range of flows along the Longview and Kelso levees due to the fixed backwater elevation at the Columbia River.

The largest step in the existing condition backwater profile occurs at a restriction in the river created between the Lexington levee and the natural feature Rocky Point near RM 7.5. Levee setback and re-terracing (removal of dredge spoils and natural fill above the 2-5 year flood stage) provides the largest opportunity for flood stage reduction along the reach. The potential for average flood stage reductions along the relocated Lexington levee range from 1.4 to 1.9 feet in the LOP range of flows.

Extension of the stage reductions achieved at the Lexington levee upstream past the Castle Rock levee is largely accomplished by removal of dredge spoils in the historic floodplain and restoration of a terrace between the 2- and 5-year flood event stages. The potential for average flood stage reductions along the Castle Rock levee range from 1.9 to 2.3 feet in the LOP range of flows.

Long-term maintenance of the setback floodplain terraces including removal of deposited material and vegetation will be required for the measure to maintain its effectiveness.

Cost Estimate

Costs were very roughly estimated by determining the land and building values of the required real estate; estimating the excavation costs and highway and railroad removal costs; and estimating the reconstruction costs of highways, railroads, setback levees, and bridge extensions. Table 8 shows the land and building values and estimates of various quantities. Figure 35 shows the location IDs corresponding to the IDs in the table. The land and building values are approximately $250 million. Table 9 shows the estimated costs for items not including the land acquisition costs. These costs are approximately $1.5 billion. The total estimated cost for expanding the floodplain is about $2 billion, and this estimate is believed to be at the low end of the range. Costs for maintaining the floodplain have not been estimated.

Table 8. Land/Building Values and Other Quantities for Expanded Floodplain Measure

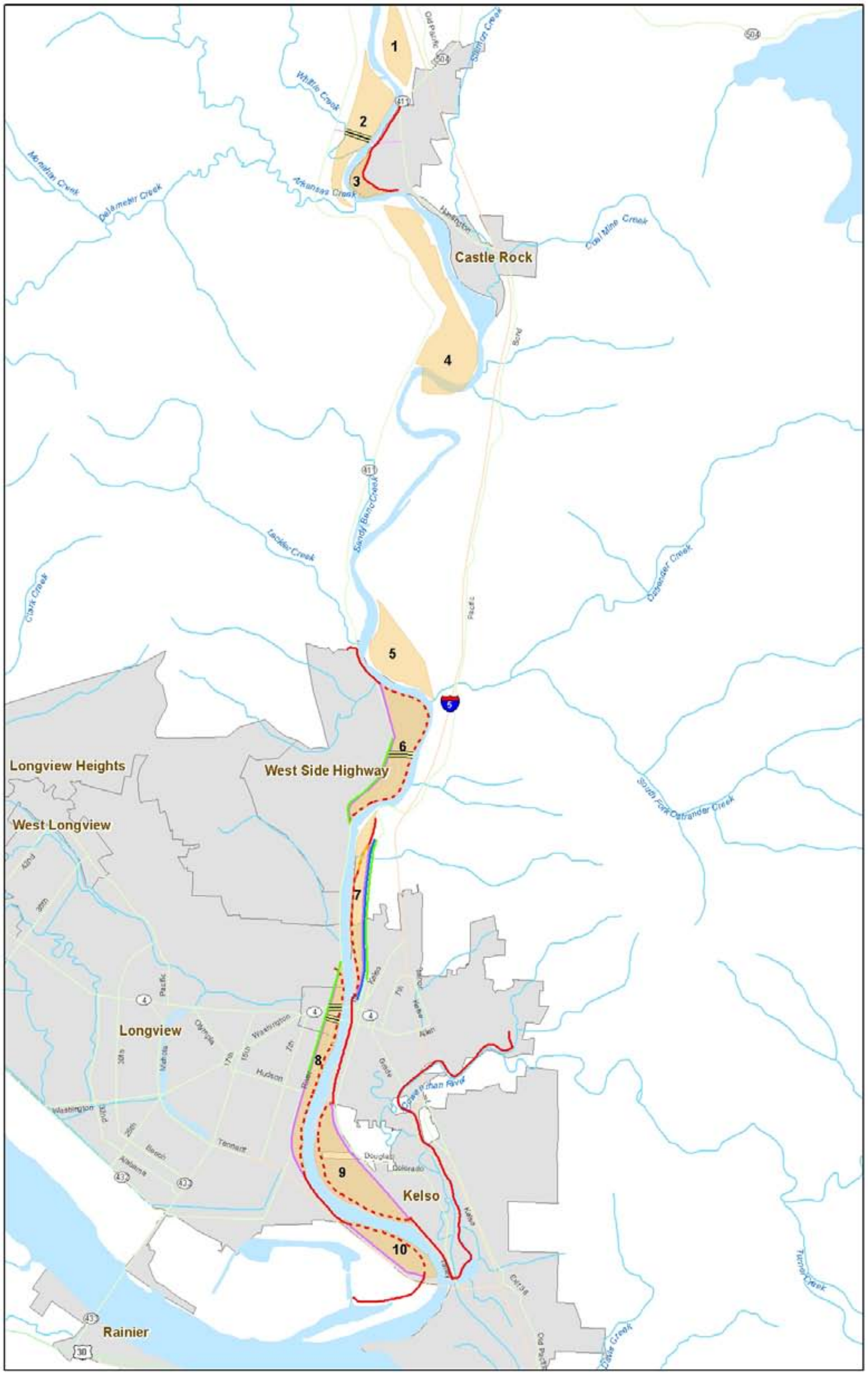
ID	LAND VALUE	BUILDING VALUE	ACREAGE	Sq. ft	MEAN removal depth (ft)	Removal Volumes (cu.ft)	Demo Levee (length, ft)	Construct levee (length, ft)	Bridge extension (ft)	Road (ft)	Rail (ft)
1	$399,430	$0	105	4371966	15	67,503,155	none	none	none	none	none
2	$4,054,920	$11,259,410	252	1787142	3	6,183,511	none	none	1,412	none	none
3	$29,420	$0	0	8875090	17	148,480,256	none	none	none	none	none
4	$4,068,030	$5,050,280	492	16740825	8	126,560,637	none	none	none	none	none
5	$888,550	$919,330	209	7811691	5	37,886,701	none	none	none	none	none
6	$32,058,310	$40,025,520	147	10310599	4	38,767,852	13,763	8,264	1,307	5,456	none
7	$4,787,070	$13,066,550	144	5174797	5	24,787,278	8238 levee; 1,451 floodwall	8,306	none	7,562	8,755
8	$24,364,290	$93,713,860	62	11512627	3	29,817,704	11,792	11,127	697 and 693	6,550	none
9	$2,801,750	$6,038,740	204	6587744	5	34,585,656	9,684	7,684	none	none	none
10	$6,568,900	$1,266,500	134	5185748	6	29,040,189	6,551	5,096	none	none	none
	$80,020,670	**$171,340,190**	**1749**								
					cu. Ft	543,612,939					
	TOTAL:	**$251,360,860**			**cu. Yds**	**20,133,813**	**51,481**	**40,477**	**4,100**	**19,567**	**8,755**

Table 9. Expanded Floodplain Cost Estimate not Including Land Acquisition Costs

COST ESTIMATE SUMMARY

	Amount	Quantity	Units	Unit Prices	
1. Mob/Demob	$226,995,206	1	LS		
2. Demolition					
A. Highway	$3,307,522	88,888	SY	$37	per SY
B. Railroad	$1,233,848	8,800	LF	$140	per LF
3. Excavation	$1,049,182,740	20,134,000	BCY	$52	per BCY
4. Reconstruction					
A. Highway	$13,712,752	88,888	SY	$154	per SY
B. Extend Bridges					
1. Lexington	$13,650,922	1,307	LF	$10,444	per LF
2. Kelso-Longview	$40,212,774	1,400	LF	$28,723	per LF
3. Railroad	$19,347,510	1,400	LF	$13,820	per LF
C. Levees	$104,238,750	825,000	BCY	$126	per BCY
D. Railroad	$4,817,912	8,800	LF	$547	per LF
Total Cost Estimate:	**$1,476,699,937**				

Figure 35. Location of Land IDs Used in Determining Land and Building Value Costs for the Expanded Floodplain Measure



Environmental Considerations

Expanding the floodplain would be positive for the environment. The river would be in a more natural condition. The floodplain would require ongoing maintenance including removal of deposited sediment and vegetation. The main channel, however, could be left undisturbed.

Real Estate Considerations

Real estate would be a major issue for this measure. To implement the measure would require the acquisition of up to 2,000 acres of land adjacent to the river.

Discussion

The analysis performed for expanding the floodplain, using an aggressive footprint for the expansion, indicates the measure has limited ability to reduce flood stages in the LOP range of flows along the Longview and Kelso levees. More promising results may be achieved for Lexington and Castle Rock. The cost of the measure is very high, much higher than that of any other measure investigated.

If the measure is to be explored further, expansion of the floodplain at the constriction between the Lexington levee and Rocky Point has the greatest potential to reduce flood stages. A limited expansion of the floodplain at Rocky Point could be investigated as its stage reduction benefits would extend upstream along the Lexington levee.

The concept of expanding the floodplain would reduce the benefits offered by the current system of levees, as some of the land with benefits would become part of the floodplain. This aspect of the measure would have to be considered if the measure were to advance for further study.

9.5.8. Measure 16 – Dikes at Mouth of Cowlitz

General Description of Strategy

Most of the sediment that deposits in the Cowlitz River deposits near the mouth. The idea of using pile dike structures to flush sediment through this reach into the Columbia River was considered.

Implementation Approach

Figure 36 shows the proposed pile dike field. The total length of the 40 dike structures that are normal to the bank is 14,000 feet. These dikes constrict the river to an average width of 350 feet. The average distance between pile dikes is about 625 feet. The total length of the two downstream dikes that are parallel to the banks is 5,500 feet. This results in a grand total length of 19,500 feet for all the pile dikes. Figure 37 shows typical details for pile dike construction.

Figure 36. Pile Dikes at Mouth of Cowlitz River

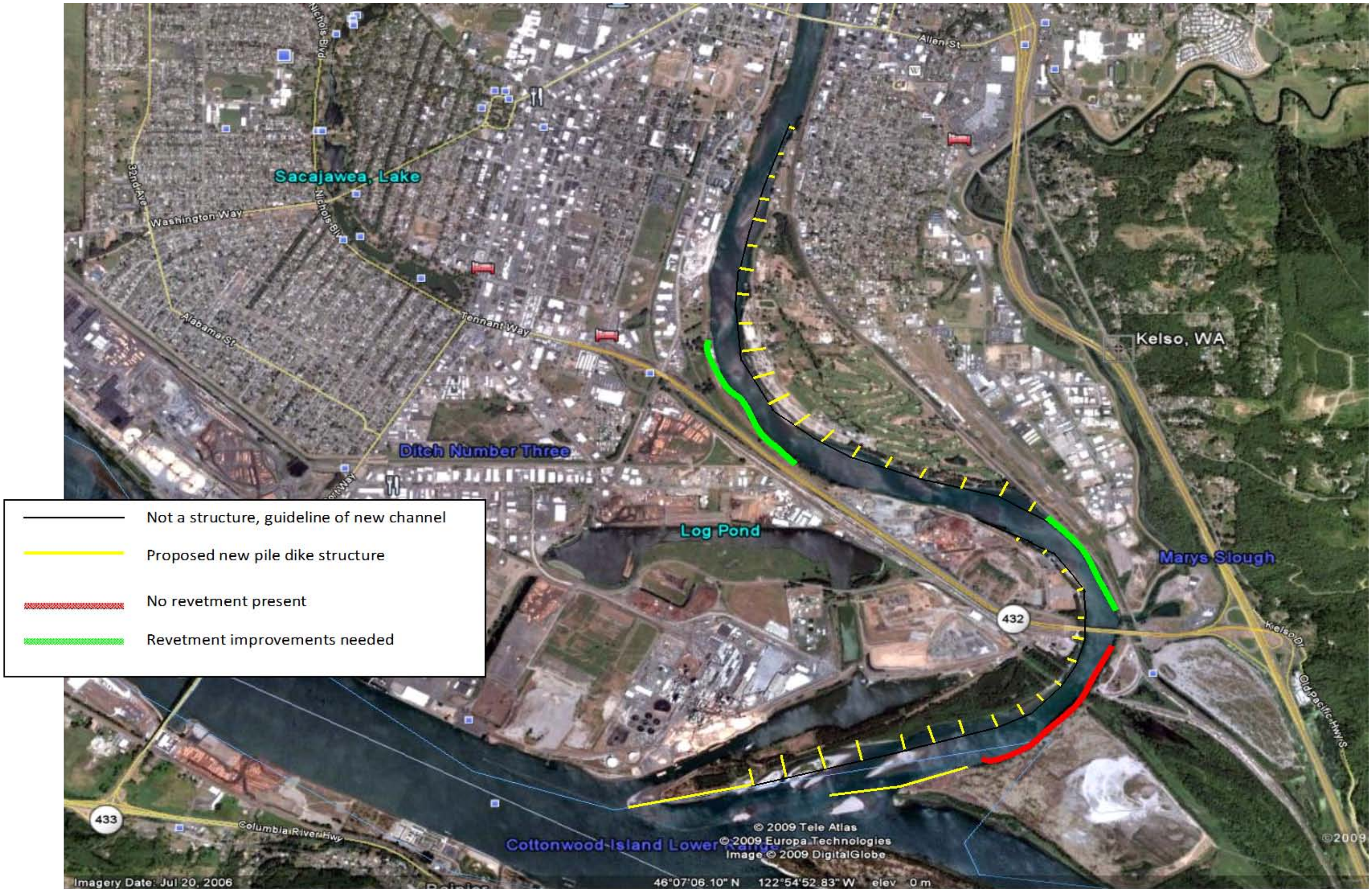
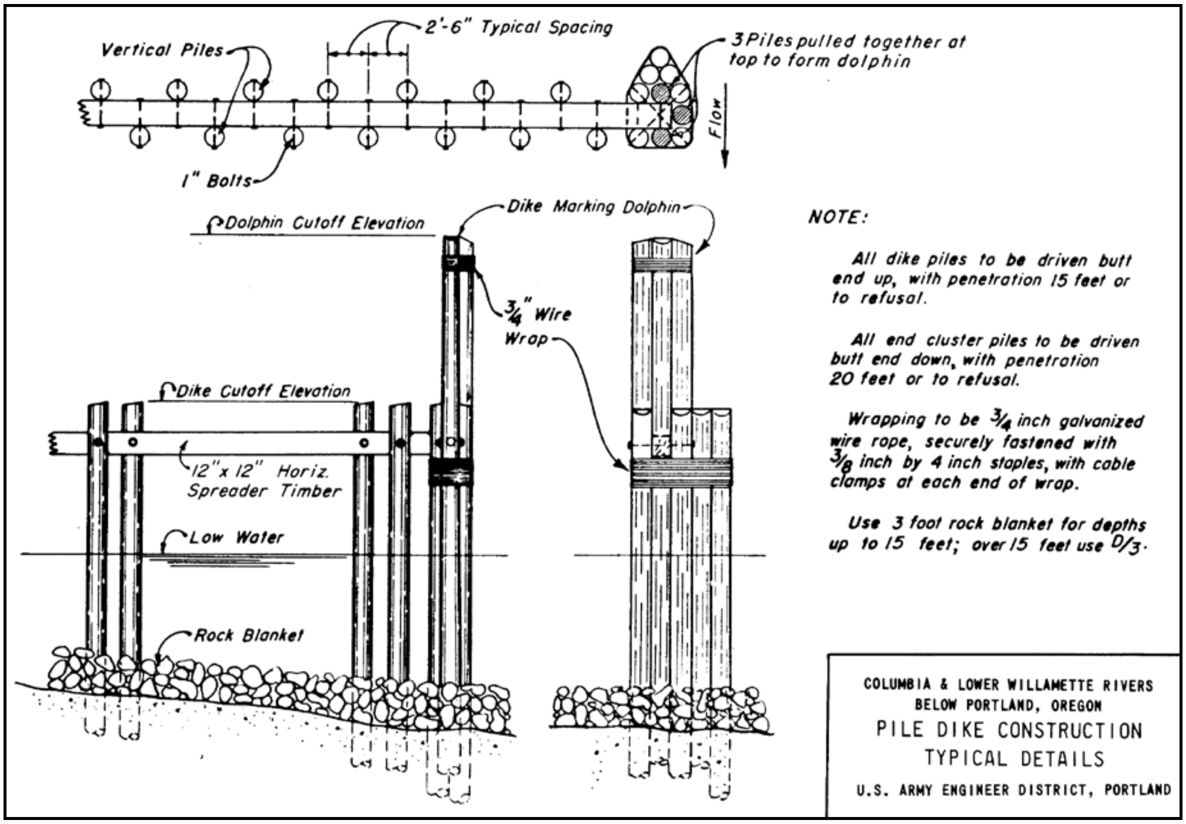


Figure 37. Pile Dike Construction, Typical Details



In order to protect the banks opposite the pile dike structures, the existing revetment along two reaches may need to be improved and a new revetment will need to be constructed along a reach where there is currently no revetment. Figure 36 shows these reaches. The lengths of the revetments potentially needing improvement and the new revetment are 11,000 feet and 8,000 feet, respectively.

Modeling Results

Appendix C includes a detailed report on the modeling of the pile dikes measure. A study was launched using a 2-dimensional model (MIKE21-C) to evaluate the impact that a dike field would have on sediment transport within the lower reaches of the Cowlitz River.

Two fully coupled 2-dimensional hydrodynamic models were created of the lower 4.5 miles of the Cowlitz River: one of the existing channel and one with a series of dikes placed throughout the lower portions of the river. Two 6-month Cowlitz River hydrographs representing high flow and typical flow water years for the Cowlitz River were run through both models to evaluate the effectiveness of the dike field in encouraging sediment movement through the lower Cowlitz.

The study area was discretized into four reaches that were compared over two years of flow within the 1992 and 1994 water year Cowlitz flow hydrographs. Sediment deposition and scour volumes were compiled and compared for the existing river configuration versus the proposed dike scenario.

Preliminary results indicate that at low flow the dike field performs similar to the existing condition, peak flow periods of typical Cowlitz flow years can transport up to 150% of the sediment compared to the existing condition, and at high Cowlitz River flows the dikes can increase sediment transport by two to three times through the system down to the mouth of the Cowlitz River.

Cost Estimate

The cost estimate for the pile dikes is based on a 2000 cost report for five similar pile dikes in the vicinity of RM 47 on the Oregon side of the Columbia River near Westport Bar. Costs from this 2000 report were increased by a factor of 1.38 based on the construction cost index history published by *Engineering News Record*. The estimated cost per linear foot of pile dike, including real estate acquisition on the banks and construction, is $1,280/foot. For 19,500 feet of pile dike, the total cost is $25 million. The ballpark costs for improving and building new revetments are $500/foot and $1,000/foot. This results in a total revetment work cost of $11.9 million. The total installation cost for the pile dike measure is $37 million.

Environmental Considerations

The current in-water work period for the mouth of the Cowlitz River from RM 0 to 2, is November through February. Above RM 2, the in-water work period is August. The pile dike and revetment work would have to occur during these periods. The potential impacts to fish from installing pile dikes in the Cowlitz River would need to be studied and, if necessary, mitigation actions would need to be identified.

Real Estate Considerations

The pile dikes would need to key in to the banks. Easements would be required to construct the bank key-ins and for long-term maintenance of the key-ins.

Discussion

Preliminary results indicate that pile dikes would reduce the volume of sediment deposition. The pile dike measure will be considered as a supplement to dredging in the Cowlitz River. Pile dikes could prove useful if the cost of installing the dikes can be offset by reduced dredging costs. Another cost that needs to be considered is Columbia River dredging. Further studies will need to estimate any increase in Columbia River dredging caused by pile dikes in the Cowlitz River.

9.6. Alternatives Development

Of the seven measures evaluated in the second screening, one measure was screened out (expanding floodplain on Cowlitz River) and the remaining six measures were grouped into alternatives for further analysis. Two measures are considered primary measures in that they have the potential to be employed as stand-alone alternatives. These measures are raised SRS (Alternative 1) and Cowlitz River dredging (Alternative 2). Secondary measures cannot be employed as stand-alone alternatives may be used to enhance the performance of the primary measures. Grade building structures, LT-1 bank stabilization, flushing flows, pile dikes, and short-term Cowlitz dredging are considered secondary measures.

Table 10 shows the 11 alternatives to be analyzed. Alternative 0 is the no action alternative. Alternatives 1a to 1d involve a raised SRS as the primary measure. Alternatives 2a to 2f involve Cowlitz River dredging as the primary measure.

Table 10. Alternatives Carried Forward for Further Analysis

Alternative	Primary Measures	Secondary Measures
0	None	Reactive measures
1a	Raised SRS	None
1b	Raised SRS	Short-term Cowlitz dredging
1c	Raised SRS	LT-1 bank stabilization
1d	Raised SRS	Both short-term dredging and LT-1 bank stabilization
2a	Cowlitz Dredging	None
2b	Cowlitz Dredging	Grade building structures
2c	Cowlitz Dredging	LT-1 bank stabilization
2d	Cowlitz Dredging	Flushing flows
2e	Cowlitz Dredging	Pile dikes
2f	Cowlitz Dredging	Some combination

Alternatives 1a to 1d. The raised SRS measure will be evaluated: (1) as a stand-alone measure; (2) supplemented by short-term Cowlitz River dredging; (3) supplemented by LT-1 bank stabilization; and (4) supplemented by both short-term dredging and LT-1 bank stabilization. If raising the SRS is selected as the preferred primary measure, it will take a few years for the measure to be implemented, and dredging in the Cowlitz River may be necessary for the interim period to manage sediment. Even with a raised SRS, it may prove beneficial to stabilize the LT-1 bank source if a large percentage of this source is depositing in the Cowlitz River.

Alternatives 2a to 2f. The Cowlitz River dredging measure will be evaluated: (1) as a stand-alone measure; (2) supplemented by grade building structures; (3) supplemented by LT-1 bank stabilization; (4) supplemented by flushing flows; (5) supplemented by pile dikes; and (6) supplemented by some combination of secondary measures to be determined. The benefits of implementing secondary measures in reducing dredging volumes will be weighed against the costs of the secondary measures.

The alternatives above involve different amounts of sediment passing through the Cowlitz River and into the Columbia River. Estimates will need to be developed as to how much sediment deposits into the Columbia River and requires dredging for navigation, so that these costs can be included in the least-cost analysis of alternatives.

The main criteria that will be used to select the preferred alternative include:

- Flood Risk. The alternative must demonstrate a reasonable assurance of maintaining the congressionally authorized levels of protection and not increasing flood risk elsewhere.
- Cost. A least-cost analysis will be performed for the alternatives.
- Environmental Impact. The impact of each alternative on the environment will be considered in the decision-making process.

10. PLAN FOR FINISHING ALTERNATIVES ANALYSIS

The following tasks are planned to finish the alternatives analysis:

- Finish study to evaluate the future sediment yield and decay rate from the debris avalanche source on Mount St. Helens. A better understanding of the future yield from the avalanche is essential for selecting the most appropriate alternative.
- The following hydraulics/hydrology and sediment transport models will be completed:
    - 1D and 2D models of SRS sediment plain. These models will be used in the evaluation of the no action alternative and in the design of the raised SRS outlet works and the grade building structures.
    - 1D and 2D mobile bed models of lower Cowlitz River. These models will be used to evaluate water surface profiles for the alternatives and in the design of the dredging prism, flushing flows, and pile dikes.
- The designs and cost estimates for the measures below will be refined and optimized. There will be close coordination between the refinement/optimization of the measures and the hydraulics/hydrology and sediment transport modeling. Due to the magnitude of the remaining work, it is planned to use AE firms to accomplish some of the tasks.
    - Sediment plain grade building structures. In addition to the 1D and 2D modeling, a pilot project is planned to build and test the concepts. Construction of the pilot project is planned for July 2010.
    - Raised SRS, including optimization of the outlet works to match the sediment release from the outlet works with the sediment transport capacity of the Cowlitz River.
    - LT-1 bank stabilization.
    - Cowlitz River dredging.
    - Flushing flows from Mossyrock Dam.
    - Pile dikes in the Cowlitz River.
- The hydraulics/hydrology and sediment transport models will be run with the optimized measures to evaluate the effectiveness of each alternative to maintain water surface profiles consistent with the congressionally authorized levels of protection through 2035.
- A least-cost analysis will be performed for the alternatives. In addition to construction costs, costs related to design, construction supervision and administration, operation and

maintenance, environmental mitigation, and dredging of Mount St. Helens sediment from the Columbia River will be included.

- The environmental impact of each alternative will be assessed. The NEPA process will be followed as discussed in Section 12.
- An alternative will be recommended and described in the final alternatives analysis report. As this report will be a decision document, the proper external independent review process will be applied.
- Appropriate technical and policy reviews will be completed throughout plan development.

11. INTERIM MEASURES

As described in Section 9, it will be 2 to 5 years before the full set of long-term plan measures could be implemented. This section describes measures that have been, will be, or may be implemented in the interim to reduce flood risk on the Cowlitz River. The measures include the Castle Rock levee seepage cutoff wall, dredging at the mouth of the Cowlitz River, LT-1 bank stabilization, and increased coordination with diking districts during flood season.

11.1. CASTLE ROCK LEVEE SEEPAGE CUTOFF WALL

In 2008, the level of protection for Castle Rock was estimated to be below 100 years, whereas the congressionally authorized level of protection is 118 years. In fall 2008, the Corps decided it would be prudent to improve the Castle Rock levee to return the level of protection above the authorized level. In summer/fall of 2009, a seepage cutoff wall was constructed in part of the levee to achieve this improvement.

A 1,700-foot long segment of the Castle Rock levee upstream of the Arkansas Valley Road Bridge was improved. This segment of levee was raised in 1980 after the eruption of Mount St. Helens. The levee's safe water level remains at or above the 1980 design water surface. The level of protection was estimated to have dropped below 100 years due to increases in flood stages. Two factors caused the increased flood stages: (1) increased sediment deposition in the Cowlitz River, and (2) a preliminary hydrology update showing a change in the Cowlitz River's flow-frequency relationship. The result was a 2-foot increase in stage for the 100-year event. To provide adequate factors of safety against seepage-related failure mechanisms, a 2.5-foot wide by 40-foot deep cement-bentonite seepage cutoff wall was constructed down the center of this segment of levee. The construction cost of this project was $1 million. The level of protection for the Castle Rock levee upstream of the bridge is currently 468 years.

11.2. COWLITZ RIVER DREDGING

In 2007 and 2008, the Corps dredged the lower 5.7 miles of the Cowlitz River as measured from the centerline of the navigation channel in the Columbia River. This dredging was in response to the heavy sedimentation in the river during water year 2007.

- From RM 0 to 0.6, about 2,188,000 cy of sediment was removed using a 30-inch pipeline dredge (*Oregon*) from November 2007 to February 2008.
- From RM 0.6 to 4.0, about 227,000 cy of sediment was removed using a 12-inch pipeline dredge (*Margeux*) from December 2007 to February 2008.

- From RM 4.0 to 5.7, about 246,000 cy of sediment was removed using a 16-inch pipeline dredge (Ross Island Dredge #10) from August to September 2008.

In addition, dredging was started in November 2009 using the dredge *Oregon* from RM 0 to 0.6. The estimated dredge volume is 1,700,000 cy.

Dredging the mouth of the Cowlitz benefits the lower part of the river to some upstream extent as the channel bed adjusts to the deepened sump created by dredging. At this point in time, the exact upstream extent of the benefit is unknown; however, it is believed that the channel bed along the lower parts of the Kelso and Longview levees is lowered due to the adjustment caused by dredging at the mouth.

11.3. LT-1 Bank Stabilization

The dredge disposal site on the right bank at LT-1 is currently undergoing erosion by the Toutle River. By comparing aerial photos, the estimated erosion volume from 1999 to 2006 was 200,000 cy or approximately 28,800 cy per year on average. The portion of the bank that is medium sand and coarser, as well as some fine sand, is likely depositing in the Cowlitz River. It may turn out that the LT-1 bank is a significant enough source of sediment that it would be cost effective to stabilize the bank. If so, the bank stabilization approach described in Section 8.5 could be employed. The timeframe for implementing bank stabilization, if it proves cost effective, could be in summer 2011.

11.4. Coordination with Diking Districts

The following activities are in place and will be continued:

- Between flood seasons, the Corps will update the level of protection estimates based on any changes in the Cowlitz River's channel conveyance due to sedimentation. A meeting will be held with the diking districts to discuss the levels of protection.
- During the annual operation and maintenance (O&M) inspection of each levee, the Corps inspector and the diking district will review the district's flood preparedness, including availability and condition of emergency supplies and equipment, and the district's written flood response plan.

Coordination with diking districts will be increased by adding Cowlitz County to the Portland District's Emergency Management list of specified Emergency Operation Centers. This addition will ensure that the Portland District has a liaison dedicated to Cowlitz County for assistance during flood events.

12. ENVIRONMENTAL COMPLIANCE

12.1. OVERVIEW

Before implementation of projects or actions that may result from the Mount St. Helens Long-Term Sediment Management Plan, the Corps is required to comply with numerous federal laws and regulations. There may also be additional requirements under state and/or local jurisdictions.

All federal actions that are funded, constructed, or permitted must comply with NEPA. The District Commander is the Corps NEPA official responsible for compliance with NEPA for actions within the District boundaries. Typically under NEPA, the District will develop a draft Environmental Assessment for construction projects. The Environmental Assessment is a brief document which provides sufficient information to the District Commander on potential environmental effects of the proposed action, if appropriate, its alternatives, and for determining whether to prepare an Environmental Impact Statement or a Finding of No Significant Impact. If project impacts are known to be major, the Corps may decide to proceed with an Environmental Impact Statement without preparing an initial Environmental Assessment.

For NEPA compliance, a number of federal laws, regulations, and executive orders must be addressed under various consultation processes. The consultation process may encompass the Clean Water Act; Coastal Zone Management Act; Endangered Species Act; Fish and Wildlife Coordination Act; Magnuson-Stevens Act (essential fish habitat); several cultural resource laws including the National Historic Preservation Act, the Archaeological Resources Protection Act, the Native American Grave Protection and Repatriation Act, the Antiquities Act, and the Archaeological and Historic Preservation Act; Executive Order 11988, Flood Plain Management; Executive Order 11990, Protection of Wetlands; Comprehensive Environmental Response, Compensation, and Liability Act; Resource Conservation and Recovery Act; Toxic Substances Control Act; Federal Insecticide, Fungicide, and Rodenticide Act; and the Migratory Bird Treaty Act, as well as other federal and state laws or regulations known to impact the project area.

Consultation with appropriate federal, state, and tribal agencies regarding potential environmental effects is coordinated through the District’s Environmental Branch. Compliance and consultation includes all permitting activities associated with the Clean Water Act including Sections 401, 402, and 404. Under Section 401 of the Clean Water Act, water quality certification would be requested from the State of Washington. Cultural resource clearance would be required for construction sites and for any potential disposal areas. Endangered Species Act compliance would include interagency consultation with the National Marine Fisheries Service and the U.S. Fish and Wildlife Service on all threatened, endangered, and proposed species including terrestrial and aquatic plants and animals.

12.2. LAWS AND REGULATIONS

The following is a list of the major federal laws and Executive Orders that may be applicable to project implementation. Included with this listing are short descriptions of the various Acts. The list is not comprehensive but is provided to display some of the potential requirements that may need to be addressed before implementation of proposed projects.

National Environmental Policy Act. This Act established the national policy promoting the enhancement of the environment and the President's Council on Environmental Quality. This Act set up the procedural requirements for all federal agencies to prepare Environmental Assessments and Environmental Impact Statements. These documents contain statements of the environmental effects of the proposed federal agency actions. The procedural requirements of NEPA apply to all federal agencies in the Executive Branch.

As stated in Section 2 of the preamble, the purpose of NEPA is *". . . to declare a national policy which will encourage productive and enjoyable harmony between man and his environment, to promote efforts which will prevent or eliminate damage to the environment and biosphere and stimulate the health and welfare of man, to enrich the understanding of the ecological systems and natural resources important to the Nation, and to establish a Council on Environmental Quality."*

Endangered Species Act. This Act establishes a national program for conservation of endangered and threatened species and their habitat. In accordance with Section 7(a)(2) of the ESA, federally funded, constructed, permitted, or licensed projects must take into consideration impacts to federally listed or proposed threatened or endangered species.

Clean Water Act. This Act sets national goals and policies to eliminate discharges of water pollutants into navigable waters, to regulate discharge of toxic pollutants, and to prohibit discharge of pollutants from point sources without permits.

Clean Air Act. This Act established a comprehensive program for improving and maintaining air quality throughout the United States. Its goals are achieved through permitting of stationary sources, restricting the emission of toxic substances from stationary and mobile sources, and establishing National Ambient Air Quality Standards. Title IV of the Act includes provisions for complying with noise pollution standards.

National Historic Preservation Act. Section 106 of the National Historic Preservation Act requires that a federally assisted or federally permitted projects account for the potential effects on sites, districts, buildings, structures, or objects that are included in or eligible for inclusion in the National Register of Historic Places.

Native American Graves Protection and Repatriation Act. This Act provides for the protection of Native American and Native Hawaiian cultural items, established ownership and control of Native American cultural items, human remains, and associated funerary objects to Native Americans. It also establishes requirements for the treatment of Native American human remains and sacred or cultural objects found on federal land. This Act also provides for the protection, inventory, and repatriation of Native American cultural items, human remains, and associated funerary objects.

Magnuson-Stevens Fishery Conservation and Management Act. This Act established procedures designed to identify, conserve, and enhance essential fish habitat for fisheries regulated under a federal fisheries management plan. Federal agencies must consult with the National Marine Fisheries Service on all proposed actions authorized, funded, or carried out by the agency that may adversely affect essential fish habitat.

Fish and Wildlife Coordination Act. This Act states that federal agencies involved in water resource development are to consult with the U.S. Fish and Wildlife Service and state agency administering wildlife resources concerning proposed actions or plans.

Migratory Bird Treaty Act. This Act provides the U.S. Fish and Wildlife Service regulatory authority to protect species of birds that migrate within and outside the United States. This Act prohibits the harming, harassing and take of protected species, except as permitted by the U.S. Fish and Wildlife Service.

Bald and Golden Eagle Protection Act. This Act provides for the protection of the bald eagle (the national emblem) and the golden eagle by prohibiting, except under certain specified conditions, the taking, possession and commerce of such birds. The 1972 amendments increased penalties for violating provisions of the Act or regulations issued pursuant thereto and strengthened other enforcement measures. Rewards are provided for information leading to arrest and conviction for violation of the Act.

Comprehensive Environmental Response, Compensation and Liability Act (Superfund). This Act provides a federal "Superfund" to clean up uncontrolled or abandoned hazardous-waste sites as well as accidents, spills, and other emergency releases of pollutants and contaminants into the environment. Through this Act, the U.S. Environmental Protection Agency was given power to seek out those parties responsible for any release and assure their cooperation in the cleanup.

Executive Order 11988, Floodplain Management. This executive order requires federal agencies to consider how their actions may encourage future development in floodplains, and to minimize such development.

Executive Order 11990, Protection of Wetlands. This executive order requires federal agencies to protect wetland habitats.

Executive Order 12898, Environmental Justice. This executive order requires federal agencies to consider and minimize potential impacts on subsistence, low-income or minority communities. The goal is to ensure that no person or group of people should shoulder a disproportionate share of the negative environmental impacts resulting from the execution of this country's domestic and foreign policy programs.

Executive Order 13175, Consultation and Coordination with Indian Tribal Governments. This executive order sets forth guidelines for all federal agencies to: (1) establish regular and meaningful consultation and collaboration with Indian tribal officials in the development of federal policies that have tribal implications; (2) strengthen the United States government-to-government relationships with Indian tribes; and (3) reduce the imposition of unfunded mandates upon Indian tribes.

Analysis of Impacts on Prime and Unique Farmlands. As a result of a substantial decrease in the amount of open farmland, the Farmland Protection Policy Act was put forth by Congress. In the statement of purpose, federal programs which contribute to the unnecessary and irreversible conversion of farmland to nonagricultural uses will be minimized. It follows that federal programs shall be administered in a manner that, as practicable, will be compatible with state and local government and private programs and policies to protect farmland.

State/Local Regulations. On a case-by-case basis, state or local laws and ordinances may also be applicable to any potential project implementation. This would be based on aspects of the individual projects if any state or local permits would be required. A Hydraulic Project Approval permit is an example of a state permit that may be required for project implementation. In some cases, contractors or sponsors may be required to obtain state or local permits.

13. SCHEDULE

It is planned to continue with the alternatives analysis in 2010. The detailed implementation schedule is dependent on results of the 2010 work. Future tasks include environmental clearances, reviews, designs, plans and specifications, and construction. As the process unfolds, the Corps will continue being responsive to changing conditions on the Cowlitz River as they may impact levels of protection.

14. LOCAL COOPERATION AND FUNDING

A Local Cooperation Agreement between the Department of Army, State of Washington, and Diking Improvement Districts was established on April 26, 1986, to construct the SRS, improve levees, and perform other required actions such as dredging. All future actions necessary to maintain flood reduction benefits for the communities along lower Cowlitz River will be performed under this agreement.

This agreement states that the Federal Government will construct the necessary facilities and operate and maintain the SRS. The State of Washington will convey to the Federal Government, at no cost, all needed lands, easements, and rights-of-way required for construction of necessary flood damage reduction facilities. The State of Washington will also operate and maintain all project mitigation measures, as well as dredged material disposal sites. The Diking Improvement Districts will operate and maintain the levees.

The Mount St. Helens Sediment Control is an open Construction General project. Annual federal funding allocations are established by the President's Budget and Congressional actions.

15. REFERENCES

U.S. Army Corps of Engineers. November 1983. A Comprehensive Plan for Responding to the Long-term Threat Created by the Eruption of Mount St. Helens, Washington. Portland District, Portland, OR.

U.S. Army Corps of Engineers. December 1984. Mount St. Helens, Washington Feasibility Report and Environmental Impact Statement, Toutle, Cowlitz and Columbia Rivers Vol. 1 and 2. Portland District, Portland, OR.

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U.S. Army Corps of Engineers. April 2002. Mount St. Helens Engineering Reanalysis, Hydrologic, Hydraulics, Sedimentation, and Risk Analysis Design Documentation Report. Portland District, Portland, OR.

U.S. Army Corps of Engineers. October 2009. Toutle/Cowlitz river Sediment Budget. Portland District, Portland, OR.

Appendix A
Sediment Evaluation Team Report

This report documents the comments made by the Sediment Evaluation Team (SET) at the conclusion of the 12-15 May 2009 meeting in Portland, OR. The meeting consisted of informative presentations by the Corps Portland District Product Delivery Team (PDT) on 12 May, a field trip on 13 May, and group discussions on 14-15 May. The SET members include:

- Jon Major, U.S. Geological Service, Cascades Volcano Observatory
- John Pitlick, University of Colorado
- Kurt Spicer, U.S. Geological Service, Cascades Volcano Observatory
- Andrew Simon, U.S. Department of Agriculture, Agricultural Research Service
- Colin Thorne, University of Nottingham
- Peter Wilcock, Johns Hopkins University

The SET offered the following 17 comments.

Comments as offered by Jon Major, John Pitlick, Andrew Simon, and Kurt Spicer

The SET members listed above met briefly after the meeting on May 14, 2009 to discuss data sources and techniques that could be used to evaluate sediment yields from the debris avalanche. Data sources are as follows:

- Cross sections;
- LiDAR-based topography;
- Bed material samples;
- Regional hydrology; and
- SRS accumulation volumes.

Comment 1: Techniques that could be used to constrain long-term estimates of sediment yield include:

- Analysis of cross section and/or LiDAR data to evaluate serial trends in erosion/deposition.
- The volume of sediment eroded from the debris avalanche needs to be coupled more precisely (quasi-annually) to the volume of sediment behind the SRS.
- Grain sizes of sediment available vs. sediment deposited could be compared.

  ⇒ **Comment 1: Full SET agreement.**

Comment 2: The PDT needs to provide more data to support the results presented in the draft sediment budget. Be more transparent; describe more clearly the techniques and assumptions used to determine trends. Given the feedback over the course of the SET meeting, consider alternative approaches for estimating sediment yields from the debris avalanche.

<u>Comment 2 discussions</u>: SET members reminded the PDT to eliminate the "overlap" and evaluate each reach's sources and sink explicitly, and suggested that, wherever possible, figures should be broken out by grain size.

⇒ **Comment 2: Full SET agreement.**

Comment 3: The Tower Road data are key to the analysis; however, relatively few measurements have been taken above 10,000 cubic feet per second (cfs). For reference, the 2-year flood at this location is 20,000+ cfs. The suspended sediment data set at Tower Road is biased by low-flow measurements. Consider segregating the data at flows above some threshold (~8,000 cfs?) to determine if a different rating-curve relation exists for high flows.

<u>Comment 3 discussions</u>: It was agreed that the Tower Road data is very important but others reminded the group of the considerable limitations to the data. There was agreement that the data is not complete; however, until an adequate sediment budget is developed, essential short-term decisions should use the available data.

⇒ **Comment 3: Full SET agreement.**

Comments as offered by Colin Thorne

Comment 4 – Dealing with Uncertainties: Uncertainties in sediment impact prediction and management are large, although this does not preclude addressing and managing sediment-related problems. Uncertainties may be dealt with by identifying the sources of uncertainty, assessing the impacts of uncertainties on model outputs, and then deciding whether levels of uncertainty are acceptable, unacceptable or tolerable. Where uncertainty is unacceptable, steps must be taken to reduce uncertainty to a level that is acceptable or at least tolerable. Steps might involve additional data collection or enhanced modeling. However, it is only justified to invest additional resources provided that the extra effort will lead to uncertainty becoming tolerable. Throughout, it must be recognized that uncertainty cannot be eliminated but only reduced to a tolerable level.

A wide range of uncertainty analysis methods are available and care must be taken to select methods appropriate to the type of data and its analytical application. The first step in uncertainty analysis is to produce a table of the sources of data, associated uncertainties and steps that might be taken to reduce uncertainties where these are found to be unacceptably high.

<u>Comment 4 discussions</u>: It was noted that the Corps, through its risk analysis, will have to define tolerable uncertainty or acceptable uncertainty.

⇒ **Comment 4: Full SET agreement.**

Comment 5 – Extreme Normal Events: The 29-year record of storms and discharges that have occurred since the 1980 eruption provides a range of events in terms of magnitude and geomorphic effectiveness. However, it may not include extreme events with low frequencies of occurrence that, although unlikely, could occur between now and 2035.

To account for the possible impacts of such extreme events, it may be prudent to perform a simulation which replaces the most extreme runoff year with a more extreme one that represents the worst case that might reasonably occur under normal conditions.

While the impacts of such an extreme event on overall conditions during the project period are unlikely to differ markedly from those for the same period without the extreme event, it should be investigated whether the outcome might be to expose people and property in the lower Cowlitz valley to unacceptable flood risk for any significant period during or following the event.

Comment 5 discussions: It was noted that mud flows with a precedent would fall into this category.

⇒ **Comment 5: Full SET agreement.**

Comment 6 – Catastrophic Events: In addition to extreme normal events that may be envisaged through extrapolation of known probabilities of rainfall intensity/duration, runoff, and sediment yield, there also exists the possibility that an event with truly catastrophic consequences might occur despite the fact that it is of exceedingly low probability. For example, the basin might experience a major seismic event that leads to destabilized hill slopes throughout the debris avalanche or causes a glacier surge in the volcano's crater, leading to a massive mudflow.

While there may be nothing that can be done to manage the risks associated with such events, it is important that they are identified and described so that the project team demonstrates that they have taken care to extend their consideration of risks beyond those that may be characterized as "normal."

⇒ **Comment 6: Full SET agreement.**

Comment 7 – Other Points, SIAM Reaches: I recommend that the System Impact Assessment Model (SIAM) results for each year be shown on a map that displays each sediment reach as being a source, transfer or sink. This would allow identification of patterns in the sediment transfer system year-on-year and in relation variations in annual hydrology and sediment input from the debris avalanche. Also, a composite map showing the frequencies with which reaches act as sources, transfers, and sinks should be produced to identify reaches that persistently operate in a particular manner and those that are more variable through time.

Comment 7 discussions: The group felt that this may assist in identifying sediment sinks for various reaches. Others wished to clarify that whenever supporting/comparative data and local knowledge are available they should be used to provide ground-truth.

⇒ **Comment 7: Full SET agreement.**

Comment 8 – SRS Sediment Plain: The sediment plain behind the SRS is behaving as a reservoir wedge deposit. In its present Phase 2 condition, it is a net storer of sediment, but there is evidence that it is exchanging coarse sediment for fine, resulting in its acting as a source for sand and a sink for gravel. This is unfortunate as it is sand that generates problems for flood risk

reduction in the lower Cowlitz. It is recommended that preliminary investigations (based around concepts for active sediment management and 2-D modeling) be performed to identify whether it might be beneficial and feasible to enhance the behavior of the area as a functional floodplain. The aim would be to induce the deposition of sand in the area in the manner that alluvium is stored on natural floodplains through vertical accretion. Preliminary discussions have already taken place with Dr Gessler and he is of the opinion that the existing 2-D model could be used for this purpose. However, prior to embarking on modeling, outline calculations should be performed to establish whether the potential for sand storage in the area would represent a tangible benefit to flood damage reduction in the lower Cowlitz valley.

Comment 8 discussions: Some felt that it would be possible to test the impact of the SRS sediment plain using the planned 2-D sediment computer model and it may be possible to do active floodplain management, which might be considered as a new potential measure.

⇒ **Comment 8: Full SET agreement.**

Comment 9 – Going beyond 2035: Based on current trends in sediment yield, it is reasonable to predict that sediment loadings in the Toutle-Cowlitz system will persist at levels between 5 and 10 mcy per annum beyond 2035. In this case, an analysis should be performed to indicate just how long it may take for sediment yields to decay to pre-eruption levels, or at least to levels that do not require on-going management actions to prevent them from impacting flood damage potential in the lower Cowlitz valley.

It is therefore recommended that long-term modeling be performed to establish the total yield and timescale for decay of sediment yields from the debris avalanche and, perhaps, the basin as a whole. This might be achieved using a landscape evolution model such as Bryce 3D, a cellular, coupled hillslope-stream model such as CAESAR (T. Coukthard, Hull University, UK), or a process-based channel evolution model such as the CONCEPTS model (NSL-ARS).

The outcomes of long-term modeling may be useful for providing a better context for sediment management and help with developing the foresight necessary to avoid making decisions now that might be regretted in the future because they reduce the capability of future sediment managers to design and implement management actions that are sustainable.

Comment 9 discussions: There was considerable discussion about long-term scenario modeling and its importance for consideration in decision making. Others wished to remind the group that this type of modeling should be as grounded as possible in the available measurements. It was asked whether the modeling would be worth pursuing given that it would not be possible to have it done by December 2009. The group suggested that the modeling would be important, particularly in considering the impacts of climate change; however, it may not be a high priority in the near-term but should be completed to inform long-term decisions. The group also suggested that, at a minimum, the Corps should continue to consider the potential long-term implications of the measures before moving forward.

⇒ **Comment 9: Full SET agreement.**

Comment 10 – Climate Change: Global warming has implications for sediment management beyond 2035 that should be assessed in order to ensure that decisions made today are consistent with the principle of precaution and avoid painting future sediment managers into corners. Regional climate change models for the Pacific Northwest could be applied in ensemble, scenario models suitable for the strategic planning of future flood risk management (including sediment management). Scenarios should include both climate change and alternatives for socio-economic development in both protected and non-levee areas of the Cowlitz floodplain.

Comment 10 discussions: This comment is similar to comment nine but in addition suggests that long-term socio-economic development in the Cowlitz flood plain should be considered in decision making.

⇒ **Comment 10: Full SET agreement.**

Comments as offered by Peter Wilcox

The information given in the *Cowlitz-Toutle River Watershed Sediment Budget* by Biedenharn Group (May 2009) was not sufficient to effectively review the assumptions and methodologies used in developing the sediment budget. The remarks below follow on subsequent presentations and discussions with the authors and Corps staff. At the broadest level, there are three concerns regarding the overall approach used in developing the sediment budget.

Comment 11: The largest term in the budget, the upstream sediment supply, was calculated as a residual. A budget residual inherently includes the error in developing the budget. Because the residual term is so large, it is difficult to assess the budget error, as well as the actual magnitude of the upstream sediment supply. This last point is critical because this sediment source – the Mount St. Helens debris avalanche – is the particular focus of interest. Information is available to estimate the spatial and temporal trends of erosion from the debris avalanche. This information is needed as the basis for understanding the trends in upstream sediment supply, evaluating the controlling mechanisms, and evaluating how erosion rates may change into the future. Sediment supply from the debris avalanche should be independently estimated and used as input to the sediment budget.

Comment 11 discussions: It was noted that the Corps should further refine (including volume and grain size) the work presented by Paul Sclafani and incorporate it into SIAM. It was further suggested that there should be a specified input on an annual basis, even if you have to estimate data for some years, as well as a calculation of the uncertainty.

⇒ **Comment 11: Full SET agreement.**

Comment 12: The richest and most valuable sources of information have not been used in developing the sediment budget. These are the record of sediment accumulation in the SRS and the record of erosion from the debris avalanche. Further, the sediment flux record for the Toutle River at Tower Road was replaced by a proxy forecast based on the South Fork Toutle River gage during the period while the SRS was filling.

An explicit budget from the debris avalanche to the Tower Road gage should be developed. Sufficient information is available to specify all terms in the budget, thereby allowing as assessment of the uncertainty in the budget. Separate budgets should be closed for mud, sand, and gravel.

Comment 12 discussions: A suggestion was made that the Biedenharn group might consider using the Kid Valley data as a surrogate to the South Fork data.

⇒ **Comment 12: Full SET agreement.**

Comment 13: There are intermediate checks on the accuracy of a sediment budget that were apparently not used. These include calculated sediment storage in river reaches for which repeat cross-sections are available, sediment transport rates at different gages, and the SRS sediment accumulation. A sediment budget is used to not only screen alternatives, but to organize our best estimates for the purpose of making large-scale forecasts. For either purpose, it is essential to use all of the reliable available information to make as many reality checks as possible and to explicitly estimate all significant budget terms such that uncertainty in the budget can be estimated. There are three concerns about the specific application of the sediment budget.

Comment 13 discussions: It was suggested that sediment transport rates, particularly for reach from SRS to Tower Road would help address some of the uncertainty in this reach (e.g., Kid Road cross section) and help to consider the temporal aspect as well. A suggestion was also made to consider adding a paragraph to this comment which links it with the issue of uncertainty.

⇒ **Comment 13: Full SET agreement.**

Comment 14: SIAM will calculate erosion or deposition by balancing sediment supply and transport capacity calculated from reach-averaged hydraulics. There are a variety of both sources and sinks not captured by these calculations. The budget was developed with explicit sediment sources but no sediment sinks. By including sources but no sinks, the budget will overestimate deposition rates (or underestimate erosion rates).

⇒ **Comment 14: Full SET agreement.**

Comment 15: The budget model developed uses a constant (2007) channel geometry for a 20-year simulation period. This will produce persistent error in locations with progressive channel change, for which a better approach is to specify a steady change in channel geometry that approximates the historical record.

Comment 15 discussions: It was noted that this data might have to be broken down by reach and this effort would require a cost-benefit analysis as it may not yield a lot of additional information. Others noted that the overall channel geometry has likely not changed much in the last 15 years. The PDT should look at the cross sections and consider applying the comment 15 "better approach" for any apparently active reaches; however, this is a low priority.

⇒ **Comment 15: Full SET agreement.**

Comment 16: Bank sediment sources were determined by comparison of 1999 and 2006 topographic information. Supply from this source is then applied to the entire 20 year simulation and, implicitly, to the forecast through 2035. Some evaluation of the change in bank erosion with time, and the representativeness of the 1999-2006 period is needed. There is one specific concern about the use of SIAM for the present application.

Comment 16 discussions: It was noted that the bank erosion estimates did not include the banks within the debris avalanche, grain size data, and uncertainty.

⇒ **Comment 16: Full SET agreement.**

Comment 17: At present, SIAM does not incorporate mixture effects on the transport rate of different grain sizes. This seemingly obscure technical point removes nearly any meaning from the gravel transport calculations. As modeled, gravel transport is represented using the Meyer-Peter and Muller formula using a constant critical Shields number. The latter factor, combined with the large amount of sand in the system, suggests that gravel transport rates are probably grossly underestimated, possibly by several orders of magnitude. The implication is that SIAM will over predict gravel deposition in the upstream reaches. This tendency to under predict gravel transport is reinforced by the use of reach-averaged hydraulics, which will also tend to under predict transport rates.

Comment 17 discussions: Some suggested that you may be able to address this question using a 2-D model.

⇒ **Comment 17: Full SET agreement.**

Appendix B
SRS/Elk Rock Sediment Retention Volume Calculations

2007 North Fork Toutle Bed Profile

Prior to 2004, annual water year reports for the North Fork Toutle River tracked bed elevation changes 6 miles upstream of the SRS. The Corps did not publish annual water year reports between 2004 and 2007, leaving a data gap in the profile. In order to continue the profiling for water year 2007, the average channel elevations were visually chosen off LiDAR at previously determined cross-section locations. Elevation data was accessed using ArcGIS and attempted to remain consistent with previous data methods. Determined 2007 elevation points were plotted along with previous profile data in relation to their distance from the SRS (Figures B-1 to B-3).

Methods – Volume Calculations

2007 LiDAR (mosaic)

The Mount St. Helens 2007 LiDAR data is organized into multiple raster data sets. Its coverage extends upstream from the mouth of the Cowlitz River, follows the North Fork Toutle River, and terminates below the Mount St. Helens crater. The area of interest, in regards to the Mount St. Helens Long-term Sediment Management Plan, is located upstream from the SRS on the North Fork Toutle River. LiDAR coverage of this area is comprised of three separate overlapping raster data sets. In order to eliminate data duplication, the three LiDAR rasters were merged to create one continuous raster. This was done using the mosaic function in ArcMap v9.2.

Surface Creations (0, s/2, s/4, shapefile, TIN, raster)

For each potential vertical raise for the SRS, three rasters were created to represent possible depositional slope equilibriums; slope of 0, slope of s/2 or 0.006, and slope of s/4 or 0.003. Each surface was created in ArcCatalog/Map and started as a polyline shapefile with set slopes. Figures B-4 to B-7 show how specific elevations were determined in order to achieve the correct slope for each surface. The elevation of each cross section was found by using the formula: [(distance from start x slope) + starting elevation]. The surface is then converted to a TIN and then a Raster in order to achieve compatibility with the 2007 LiDAR data.

Cut/Fill Tool

"When the Cut Fill operation is performed, by default a specialized renderer is applied to the layer that highlights the locations of cut and of fill. The determinant is in the attribute table of the output raster, which considers positive volume to be where material was cut (removed), and negative volume where material was filled (added)."[1]

By comparing the 2007 LiDAR data to the slope rasters using the cut fill operation, areas of added or removed material are highlighted. Once this operation is executed, the data can be exported in the form of an excel spreadsheet and evaluated (Figures B-8 to B-13).

[1] ArcMap online help description.

Figure B-1. Data prior to 2007 taken from previous annual water year reports with added 2007 profile.

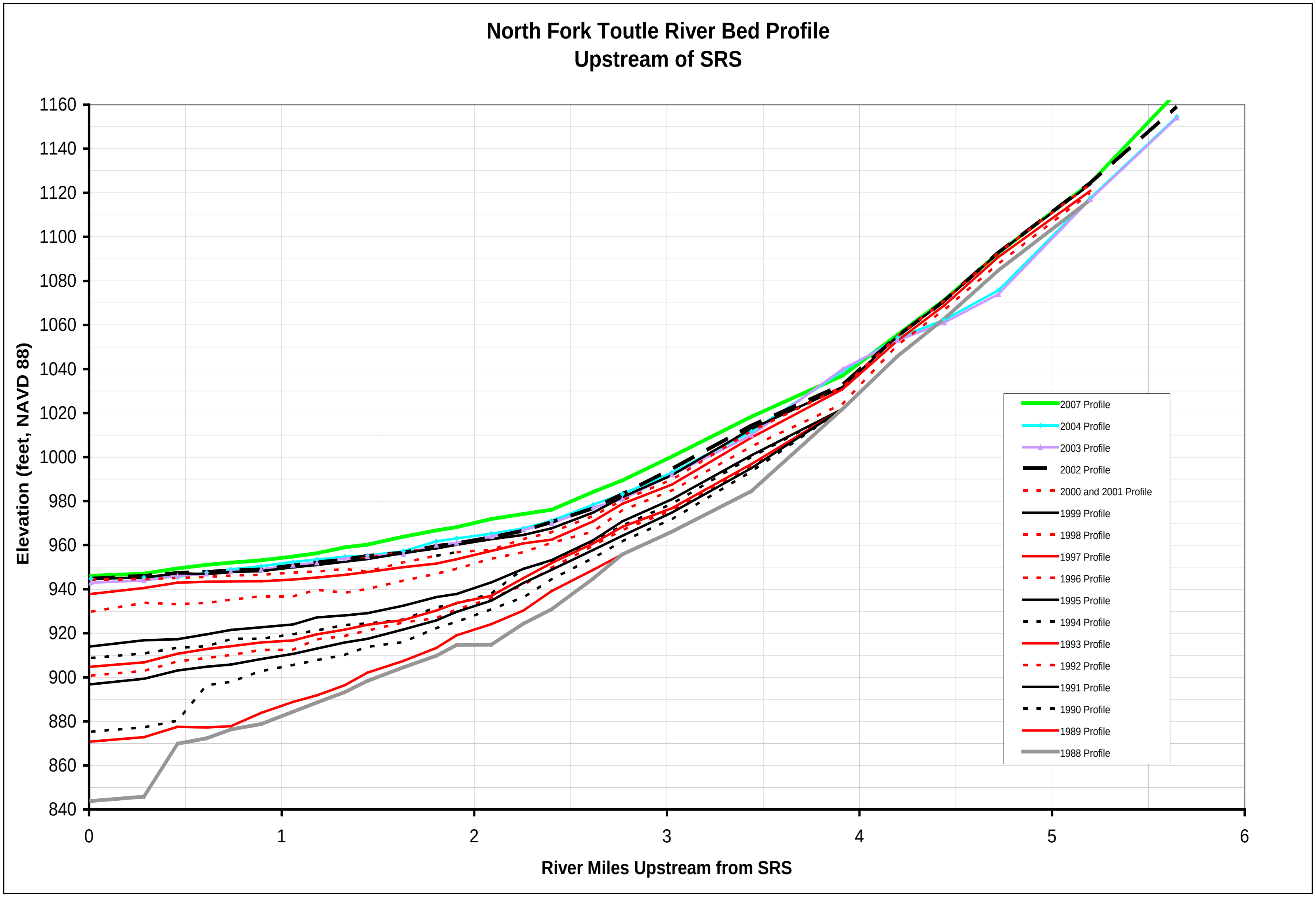


Figure B-2. Example from the 2007 LiDAR data located upstream of the SRS at cross-section 19-94. Previous data taken from 2000 water year report. The 2007 visual average line is indicated in blue.

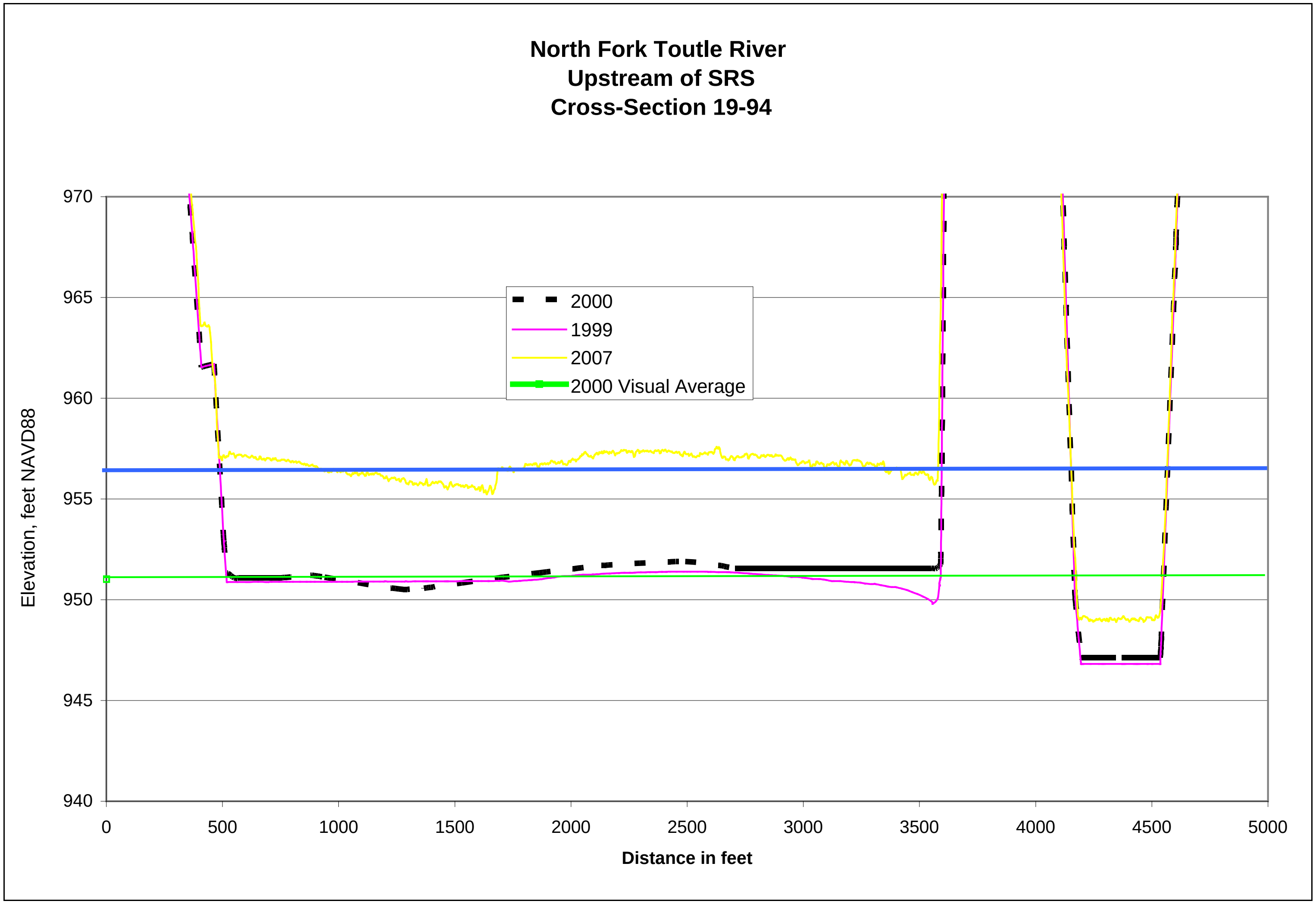


Figure B-3. Plotted bed elevations are visual averages taken from 2007 LiDAR data at predetermined cross-section locations. Plotted points of interest are at estimated river mile and elevation.

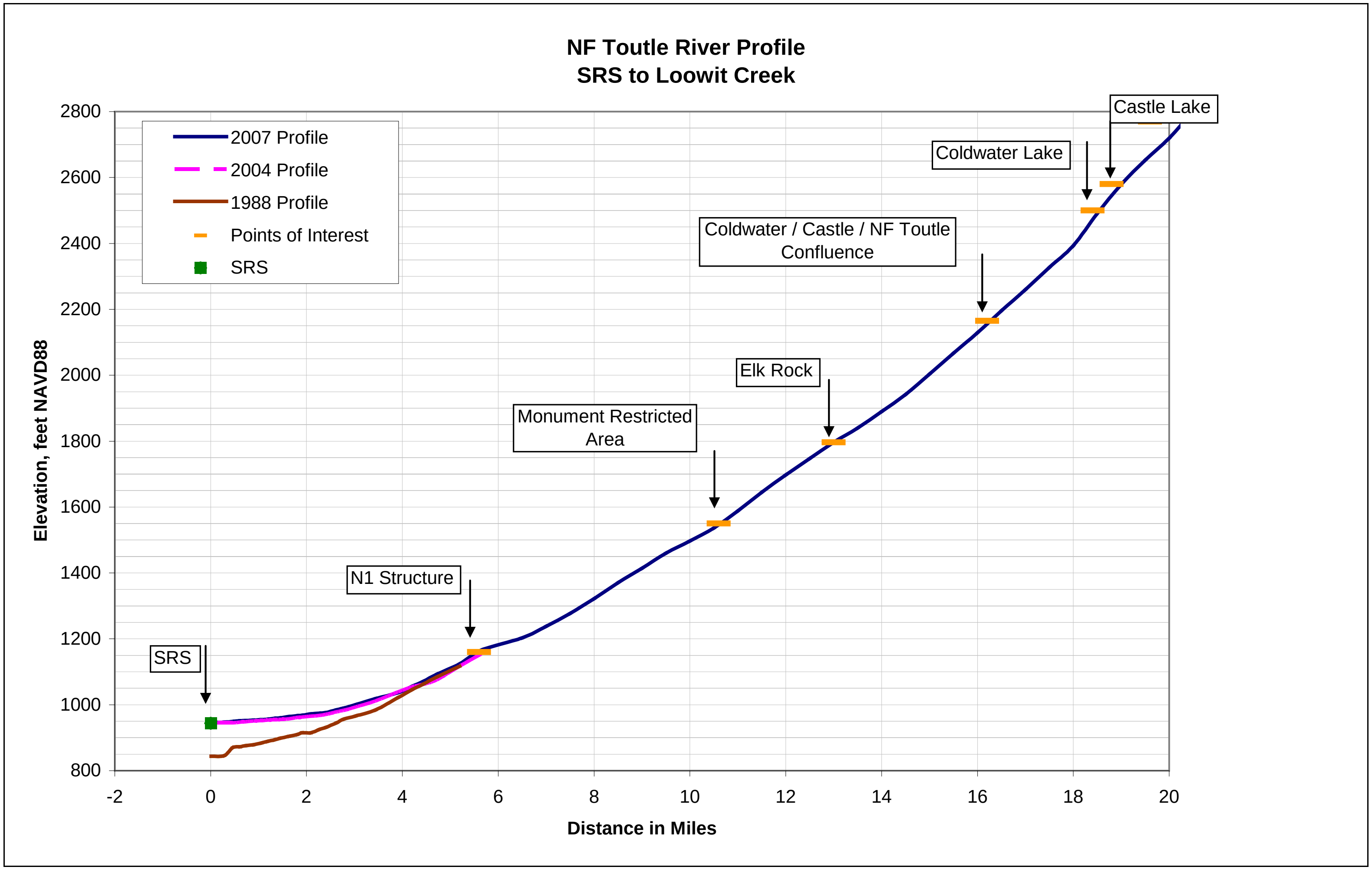
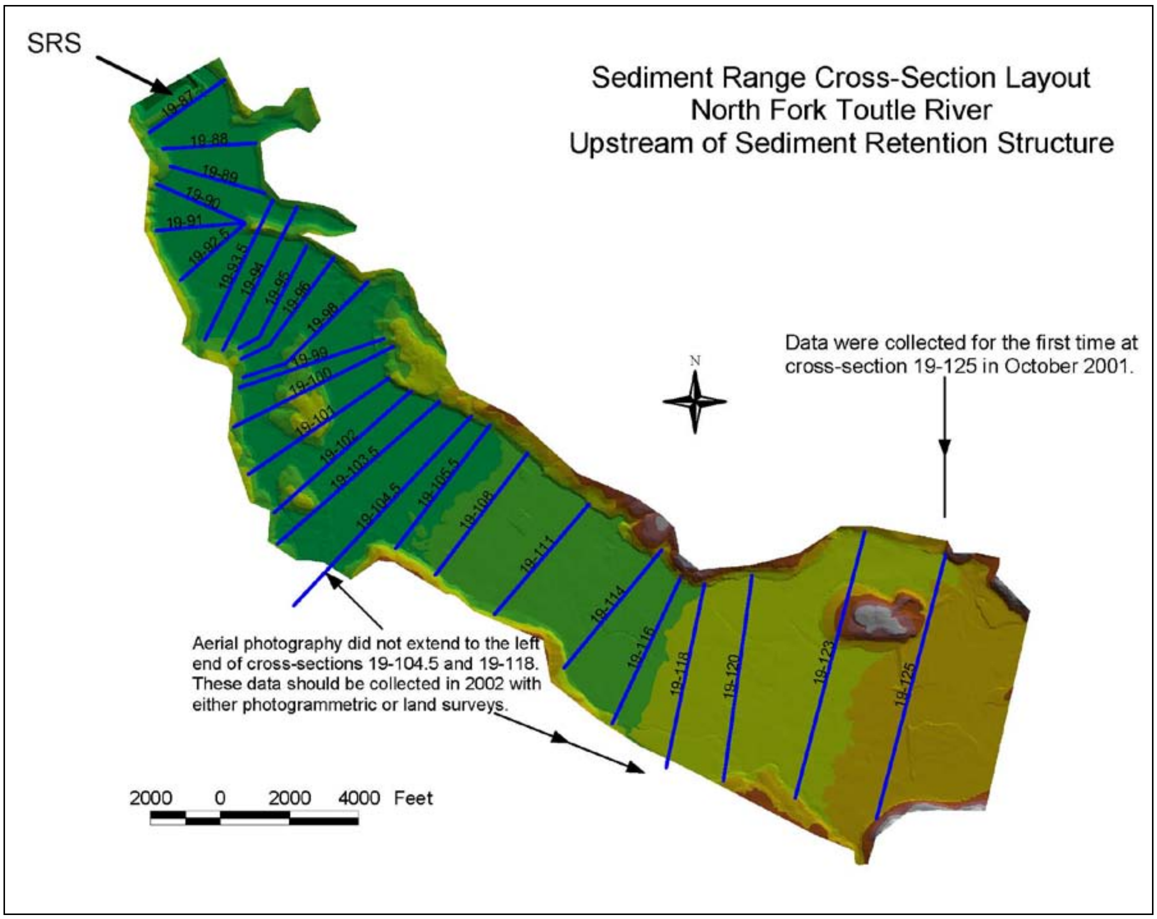


Figure B-4. Slope surfaces North Fork Toutle River upstream of SRS.

XS Number	Distance from SRS, feet NAVD 88
1	5590
2	10077
3	15988
4	20654
5	24923
6	29820
7	35000
8	40280
9	45560
10	50840



*Cross-section #1-7 locations are based on previous locations; #8-10 spaced at 0.5 miles

Figure B-5. SRS slope surface elevation spreadsheet.

Equation used to set surface elevations = (distance from SRS x slope) + spillway crest elevation.

GIS Surface	Slope	Starting Elevation, feet NAVD88	1	2	3	4	5	6	7	8	9	10
El944	0	944	944	944	944	944	944	944	944.00			
El.944_vol_1	0.003	944	960.77	974.231	991.964	1005.962	1018.769	1033.46	1049.00			
El944_vol_2	0.006	944	977.54	1004.462	1039.928	1067.924	1093.538	1122.92	1154.00			
el944_vol3	0.0108	944	1004.372	1052.832	1116.67	1167.063	1213.168	1266.056	1322.00	1379.024	1436.048	1493.072
Multi-structure concept												
El980	0	980	980	980	980	980	980	980	980.00			
EL980_vol_1	0.003	980	996.77	1010.231	1027.964	1041.962	1054.769	1069.46	1085.00			
EL980_vol_2	0.006	980	1013.54	1040.462	1075.928	1103.924	1129.538	1158.92	1190.00			
El1000	0	1000	1000	1000	1000	1000	1000	1000	1000.00			
El1000_vol_1	0.003	1000	1016.77	1030.231	1047.964	1061.962	1074.769	1089.46	1105.00			
El1000_vol_2	0.006	1000	1033.54	1060.462	1095.928	1123.924	1149.538	1178.92	1210.00			
El1050	0	1050	1050	1050	1050	1050	1050	1050	1050.00			
El1050_vol_1	0.003	1050	1066.77	1080.231	1097.964	1111.962	1124.769	1139.46	1155.00			
El1050_vol_2	0.006	1050	1083.54	1110.462	1145.928	1173.924	1199.538	1228.92	1260.00	1291.68	1323.36	1355.04
El1100	0	1100	1100	1100	1100	1100	1100	1100	1100.00			
El1100_vol_1	0.003	1100	1116.77	1130.231	1147.964	1161.962	1174.769	1189.46	1205.00			
El1100_vol_2	0.006	1100	1133.54	1160.462	1195.928	1223.924	1249.538	1278.92	1310.00	1341.68	1373.36	1405.04
el1150	0	1150	1150	1150	1150	1150	1150	1150	1150.00	1150	1150	1150
el1150v1	0.003	1150	1166.77	1180.231	1197.964	1211.962	1224.769	1239.46	1255.00	1270.84	1286.68	1302.52
el1150v2	0.006	1150	1183.54	1210.462	1245.928	1273.924	1299.538	1328.92	1360.00	1391.68	1423.36	1455.04

Figure B-6. Elk Rock slope surfaces.

XS Map Number	Table number	Distance from XS5.0, feet
5.5	1	2640
6	2	5280
6.5	3	7920
7	4	10560
7.5	5	13200
8	6	15840
8.5	7	18480
9	8	21120
9.5	9	23760
10	10	26400
10.5	11	19040
11	12	31680
11.5	13	34320

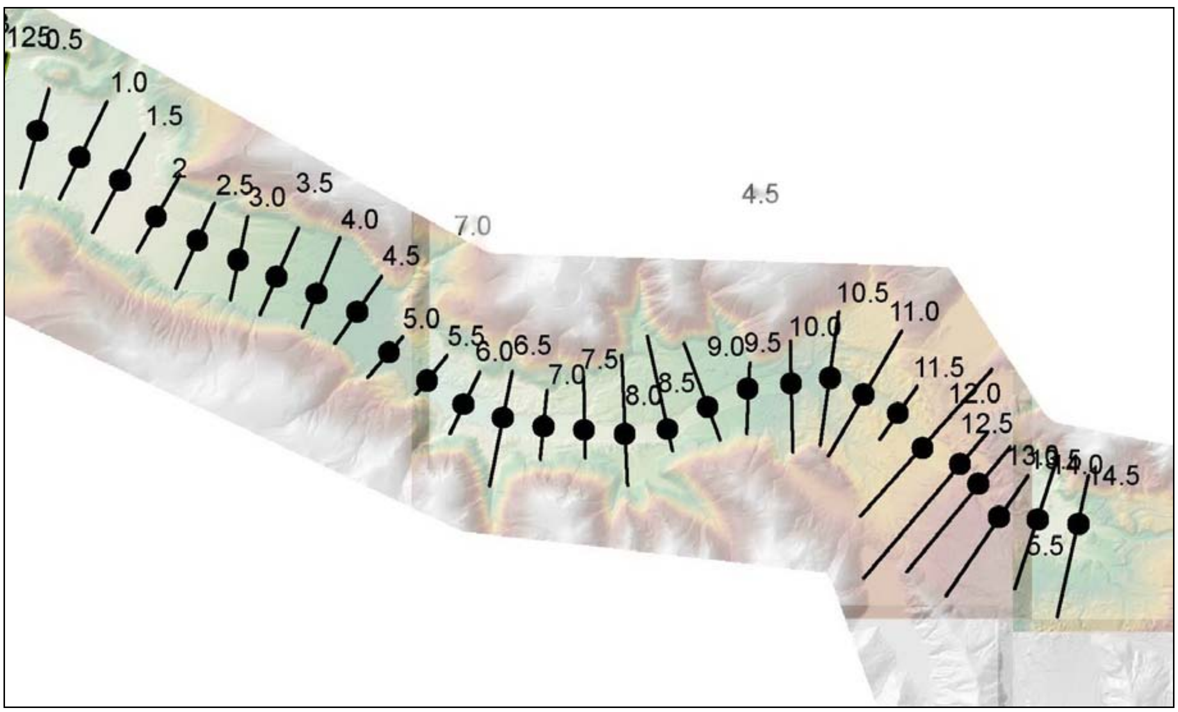
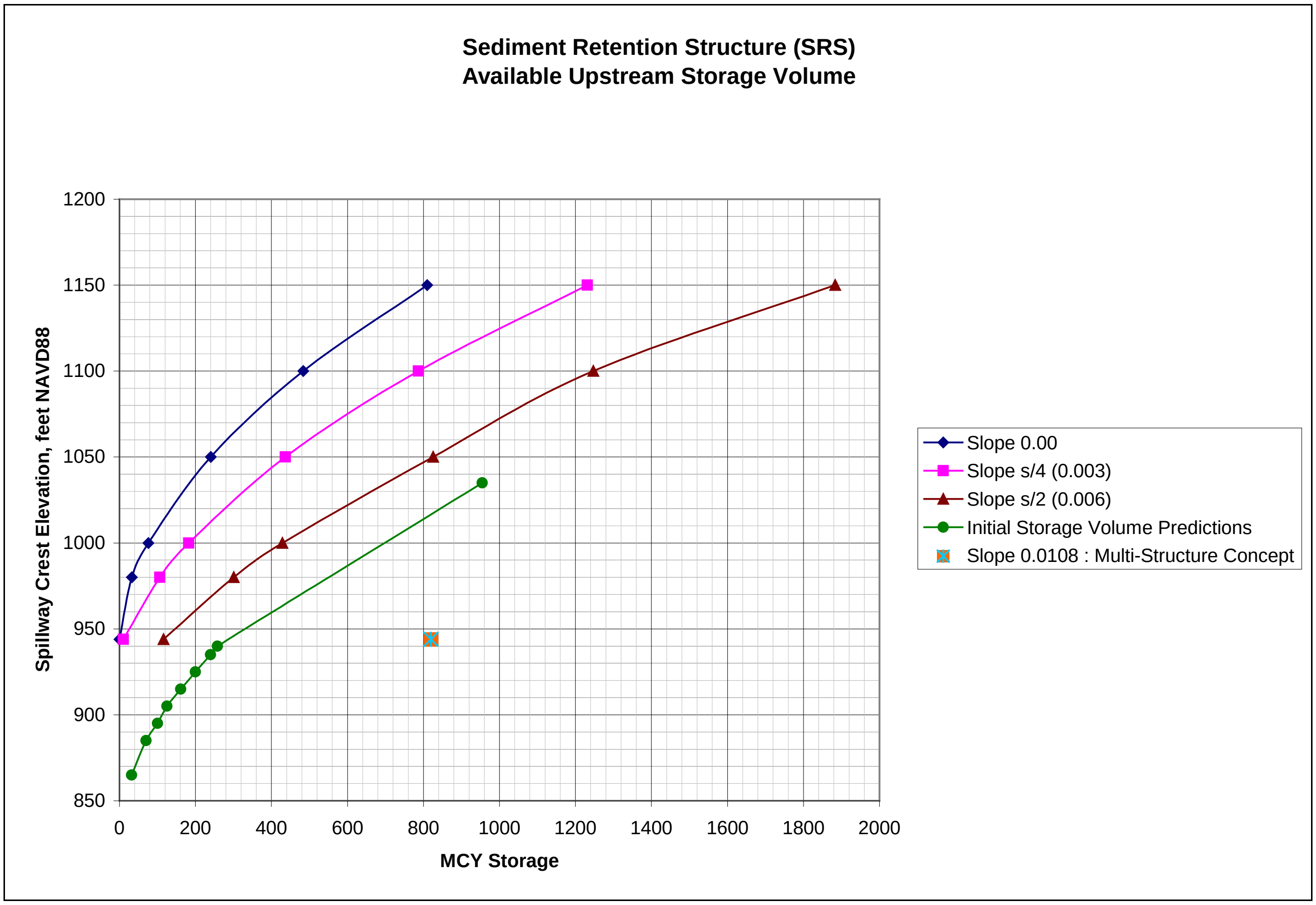


Figure B-7. Elk Rock slope surface elevation spreadsheet.

GIS Surface	Slope	Starting Elevation, feet NAVD88	1	2	3	4	5	6	7	8	9	10	11	12	13
El1700	0	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700
el1700v1	0.003	1700	1707.92	1715.84	1723.76	1731.68	1739.6	1747.52	1755.44	1763.36	1771.28	1779.2	1757.12	1795.04	1802.96
(112' structure)															
el1700v2	0.006	1700	1715.84	1731.68	1747.52	1763.36	1779.2	1795.04	1810.88	1826.72	1842.56	1858.4	1814.24	1890.08	1905.92
(112' structure)															
El1800	0	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
el1800v1	0.003	1800	1807.92	1815.84	1823.76	1831.68	1839.6	1847.52	1855.44	1863.36	1871.28	1879.2	1857.12	1895.04	1902.96
(212' structure)															
el1800v2	0.006	1800	1815.84	1831.68	1847.52	1863.36	1879.2	1895.04	1910.88	1926.72	1942.56	1958.4	1914.24	1990.08	2005.92
(212' structure)															
El1900	0	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
(312'structure)															
el1900v1	0.003	1900	1907.92	1915.84	1923.76	1931.68	1939.6	1947.52	1955.44	1963.36	1971.28	1979.2	1957.12	1995.04	2002.96
el1900v2	0.006	1900	1915.84	1931.68	1947.52	1963.36	1979.2	1995.04	2010.88	2026.72	2042.56	2058.4	2014.24	2090.08	2105.92
el2000	0	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000
(412' structure)															
el2000V1	0.003	2000	2007.92	2015.84	2023.76	2031.68	2039.6	2047.52	2055.44	2063.36	2071.28	2079.2	2057.12	2095.04	2102.96
el2000V2	0.006	2000	2015.84	2031.68	2047.52	2063.36	2079.2	2095.04	2110.88	2126.72	2142.56	2158.4	2114.24	2190.08	2205.92
el2100	0	2100	2100	2100	2100	2100	2100	2100	2100	2100	2100	2100	2100	2100	2100
el2100v1	0.003	2100	2107.92	2115.84	2123.76	2131.68	2139.6	2147.52	2155.44	2163.36	2171.28	2179.2	2157.12	2195.04	2202.96
el2100v2	0.006	2100	2115.84	2131.68	2147.52	2163.36	2179.2	2195.04	2210.88	2226.72	2242.56	2258.4	2214.24	2290.08	2305.92
el2200	0	2200	2200	2200	2200	2200	2200	2200	2200	2200	2200	2200	2200	2200	2200
el2200v1	0.003	2200	2207.92	2215.84	2223.76	2231.68	2239.6	2247.52	2255.44	2263.36	2271.28	2279.2	2257.12	2295.04	2302.96
el2200v2	0.006	2200	2215.84	2231.68	2247.52	2263.36	2279.2	2295.04	2310.88	2326.72	2342.56	2358.4	2314.24	2390.08	2405.92
el2300	0	2300	2300	2300	2300	2300	2300	2300	2300	2300	2300	2300	2300	2300	2300
el2300v1	0.003	2300	2307.92	2315.84	2323.76	2331.68	2339.6	2347.52	2355.44	2363.36	2371.28	2379.2	2357.12	2395.04	2402.96
el2300v2	0.006	2300	2315.84	2331.68	2347.52	2363.36	2379.2	2395.04	2410.88	2426.72	2442.56	2458.4	2414.24	2490.08	2505.92
el2400	0	2400	2400	2400	2400	2400	2400	2400	2400	2400	2400	2400	2400	2400	2400
el2400v1	0.003	2400	2407.92	2415.84	2423.76	2431.68	2439.6	2447.52	2455.44	2463.36	2471.28	2479.2	2457.12	2495.04	2502.96
el2400v2	0.006	2400	2415.84	2431.68	2447.52	2463.36	2479.2	2495.04	2510.88	2526.72	2542.56	2558.4	2514.24	2590.08	2605.92
el2500	0	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500
(912' structure)															
el2500v1	0.003	2500	2507.92	2515.84	2523.76	2531.68	2539.6	2547.52	2555.44	2563.36	2571.28	2579.2	2557.12	2595.04	2602.96
el2500v2	0.006	2500	2515.84	2531.68	2547.52	2563.36	2579.2	2595.04	2610.88	2626.72	2642.56	2658.4	2614.24	2690.08	2705.92

Figure B-8. SRS storage volumes.



SRS Storage Volumes (Spillway Crest Elevations in NAVD88)

	Sediment Slope			
Spillway Elevation	Flat (0)	s/4 (0.003)	s/2 (0.006)	Multi-Struture Concept (0.0108)
944	0	10	116	819
980	33	106	301	N/A
1000	76	182	429	N/A
1050	240	437	826	N/A
1100	484	787	1247	N/A
1150	810	1231	1883	N/A

*Note: Sediment Volumes given in MCY

Figure B-9. SRS estimated upstream storage capacity at spillway crest elevation 944 feet (NAVD88).

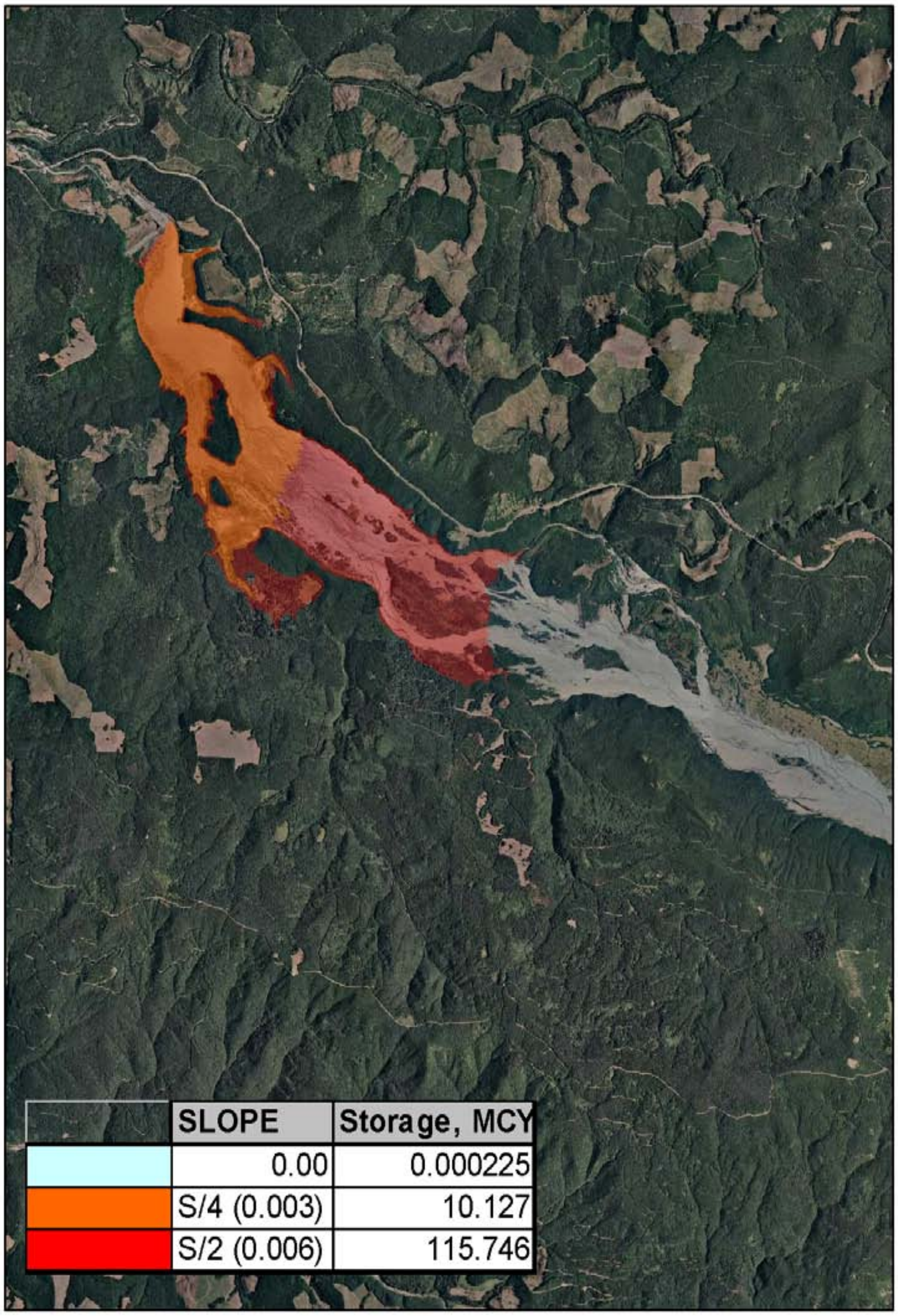


Figure B-10. SRS estimated upstream storage capacity at spillway crest elevation 1050 feet (NAVD88).

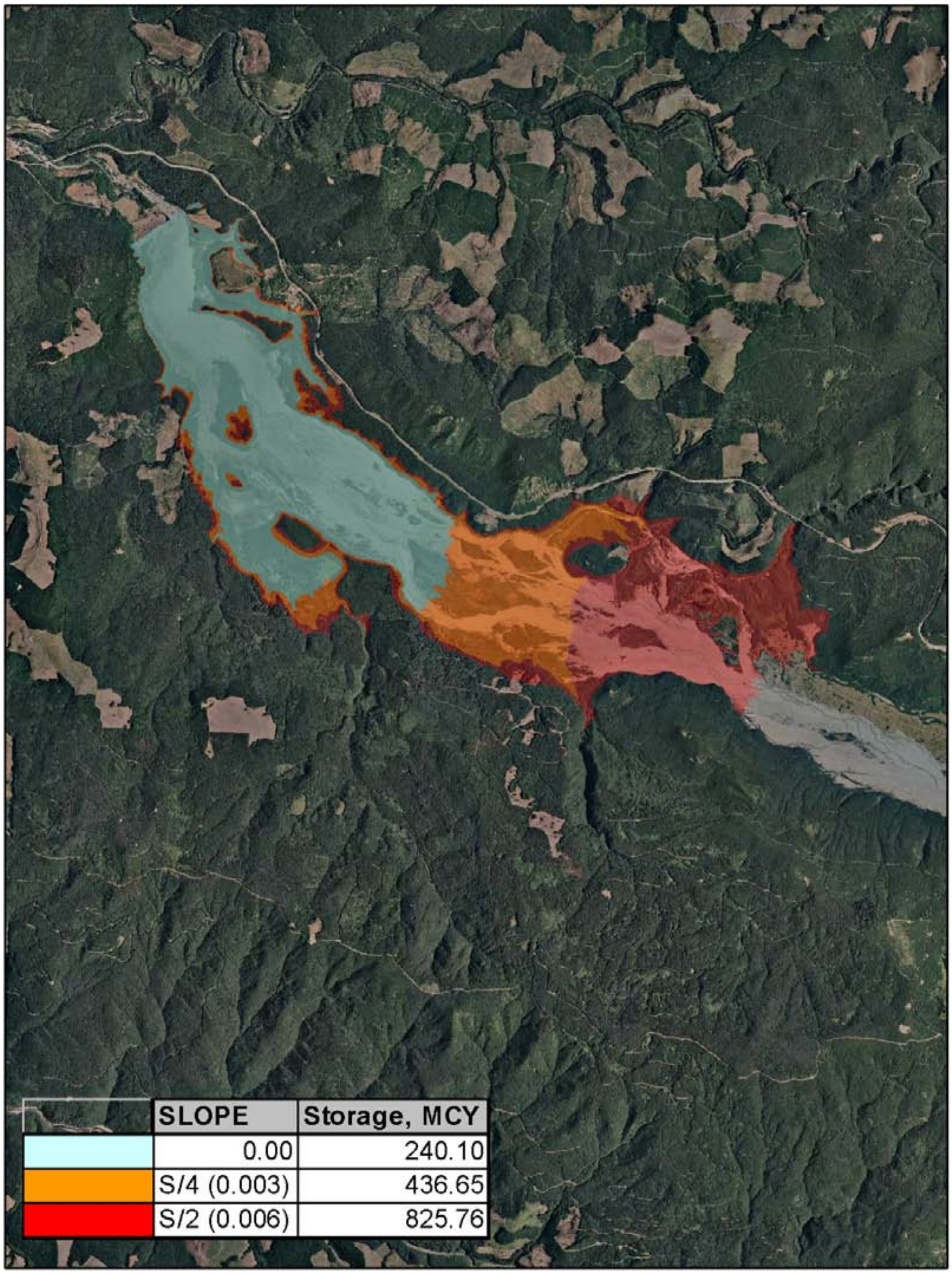


Figure B-11. Storage footprint with SRS spillway raise to elevation 1100 feet (NAVD88).

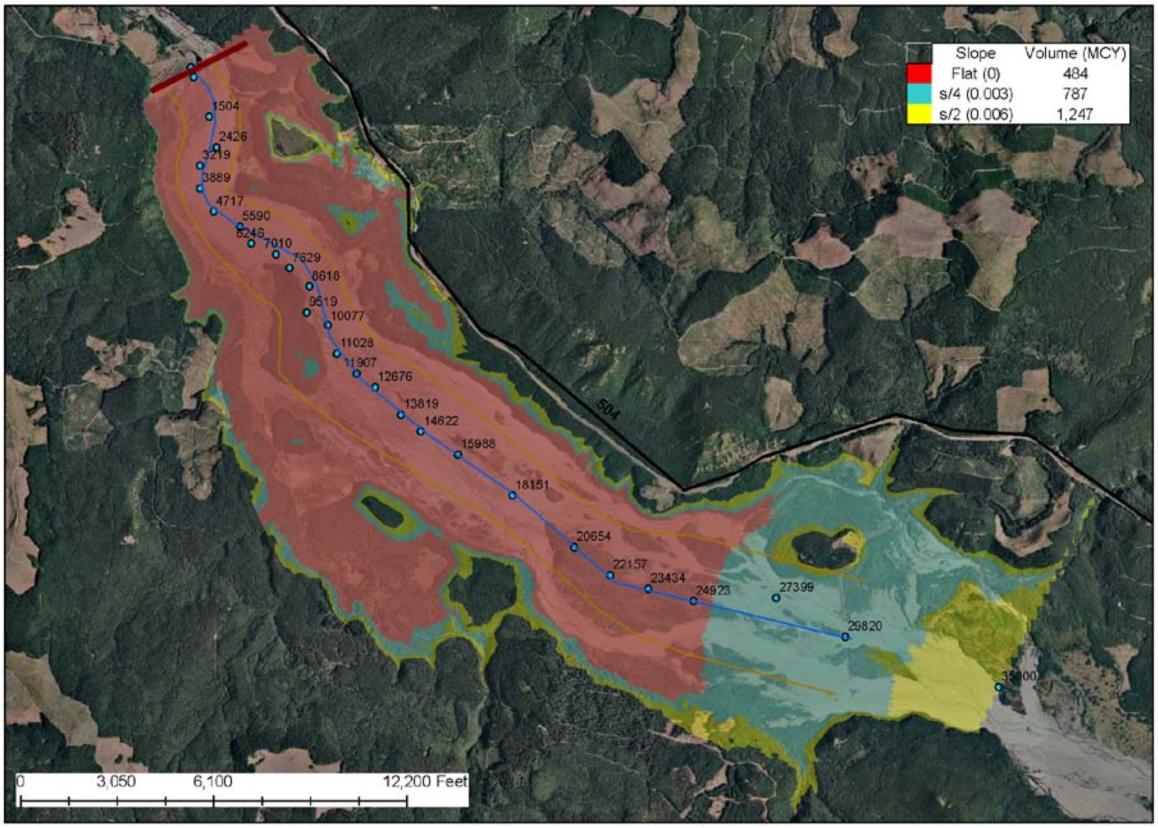


Figure B-12. SRS estimated upstream storage capacity at spillway crest elevation 1150 feet (NAVD88).

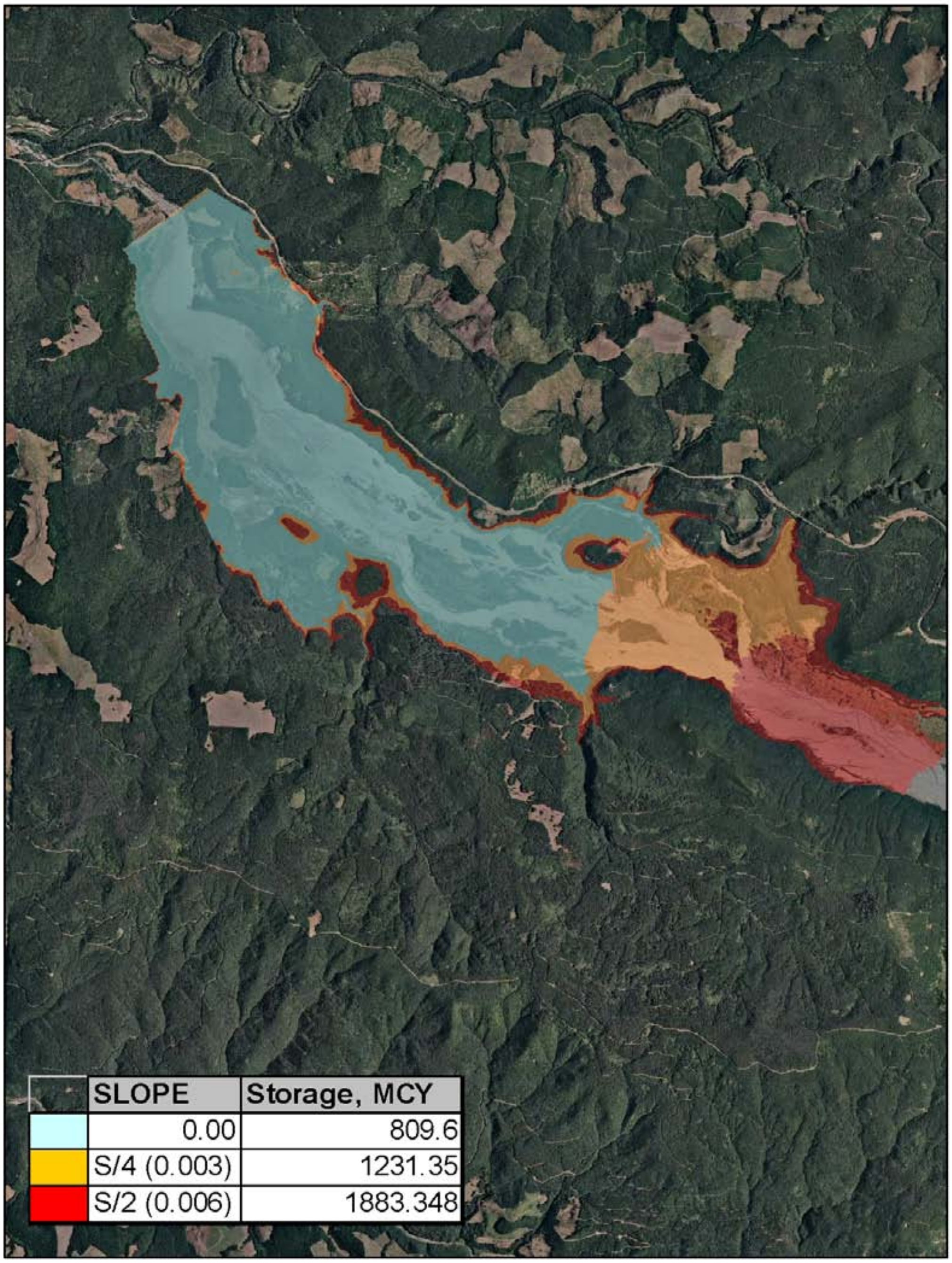
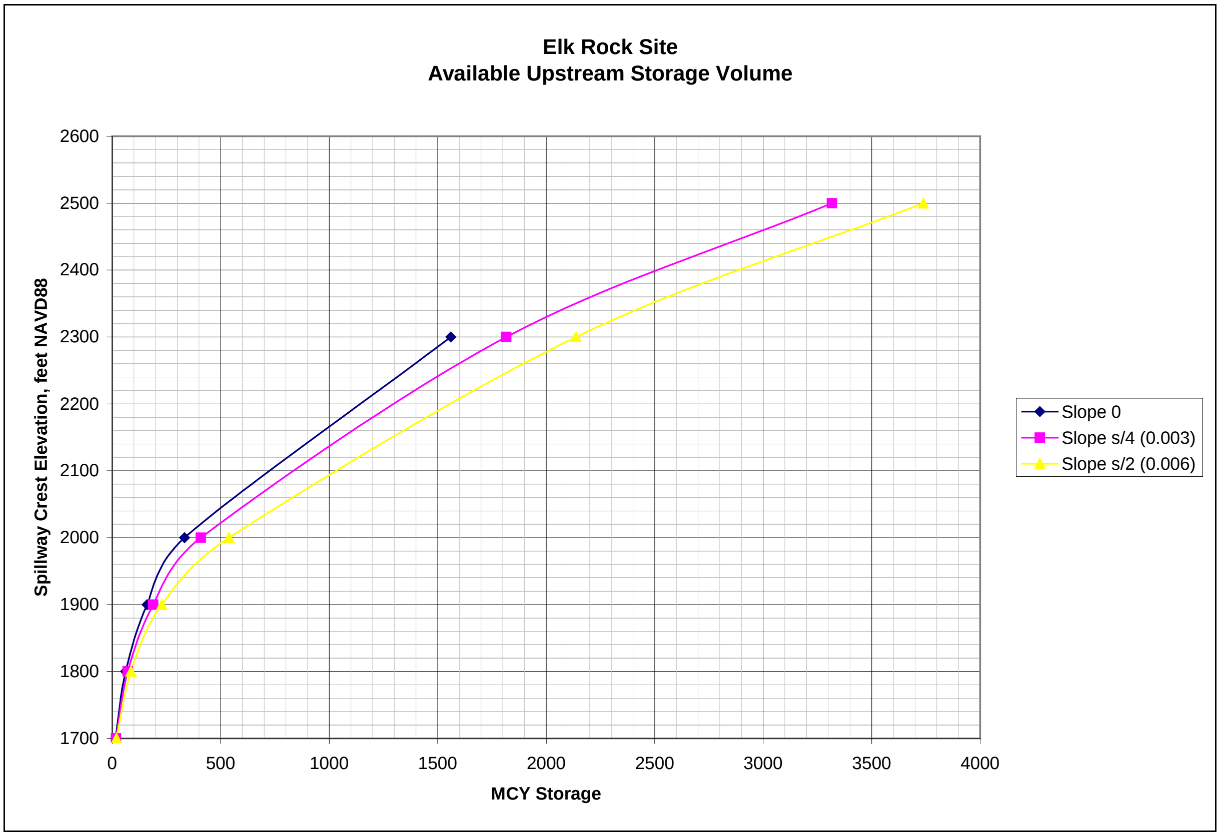


Figure B-13. Elk Rock storage volumes.



Elk Rock Storage Volumes (Spillway Crest Elevations in NAVD88)

		Sediment Slope	
Spillway Elevation	**Flat (0)**	**s/4 (0.003)**	**s/2 (0.006)**
1700	16	18	20
1800	61	72	86
1900	161	189	229
2000	333	409	538
2300	1560	1816	2138
2500		3317	3738

*Note: Sediment Volumes given in MCY

Appendix C
Performance Modeling of Select Measures

Contents:

Analysis of Measures Using the Sediment Budget – Raised SRS Grade Control Structures and LT1 Sump

1.0 Introduction and Methodology

The Toutle and Cowlitz Rivers sediment budget was developed by the Portland District, USACE in conjunction with The Biedenharn Group, LLC and is documented in *The Toutle/Cowlitz River Sediment Budget Draft Report*, [USACE & The Biedenharn Group, August 2009]. The main purpose of the sediment budget was to identify existing sediment sources, pathways, and sinks by grain class to provide a framework for identifying, evaluating, and Level 1 screening of potential alternatives. The sediment budget report contains nine annual sediment budgets for water years 1999 – 2007. These annual sediment budgets for the mouth of the Toutle River were used in a Monte-Carlo type analysis to predict a possible range of sediment loads by 2035, using randomly selected 27-year combinations of the nine annual budgets. Major sediment sources identified in the Toutle/Cowlitz sediment budget for water years 1999 – 2007 are shown graphically in Figure 1.1. Results of the sediment budget indicate that sediment output from the spillway of the SRS is the largest contributor to the watershed.

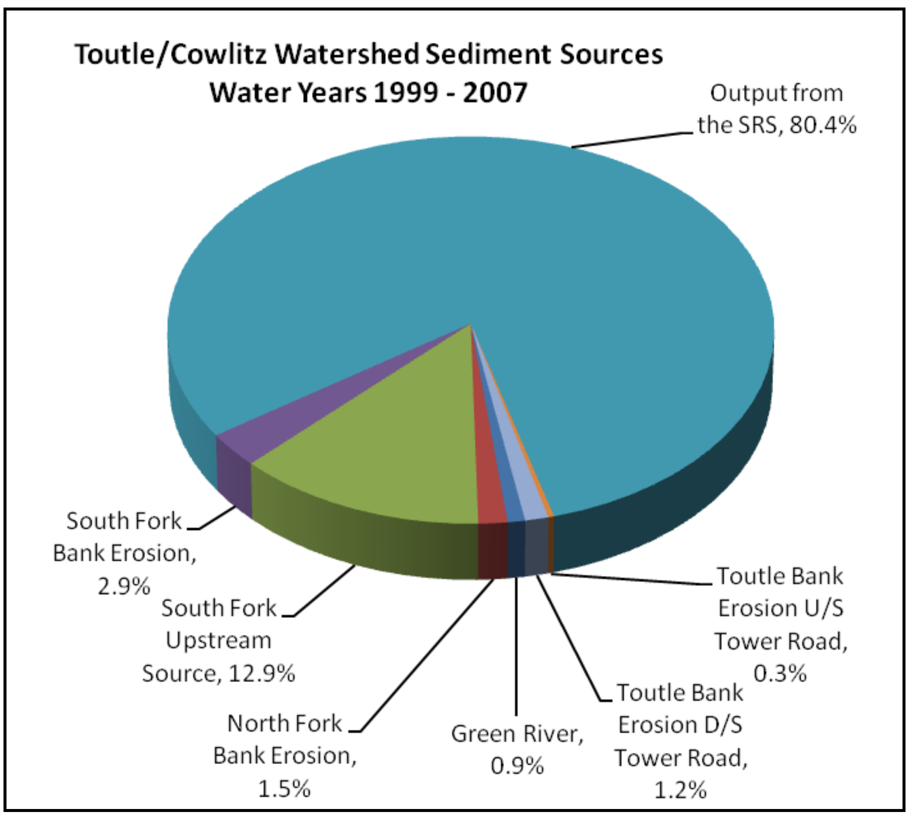


Figure 1.1 Sediment Sources to the Toutle/Cowlitz Watershed for Water Years 1999 - 2007

Evaluations of Cowlitz River bed material samples and hydraulic conditions indicate that material < 0.125 mm (silt, clay, and very fine sand) is not depositing in large quantities at the mouth and is likely moved through the Cowlitz to the Columbia River by as washload. Analysis presented in the sediment budget report indicates that material depositing in the lower Cowlitz is between 0.125 to 2 mm (fine sand to very coarse sand). Annual sediment loads at the mouth of the Toutle River by grain size are presented in Figure 1.2.

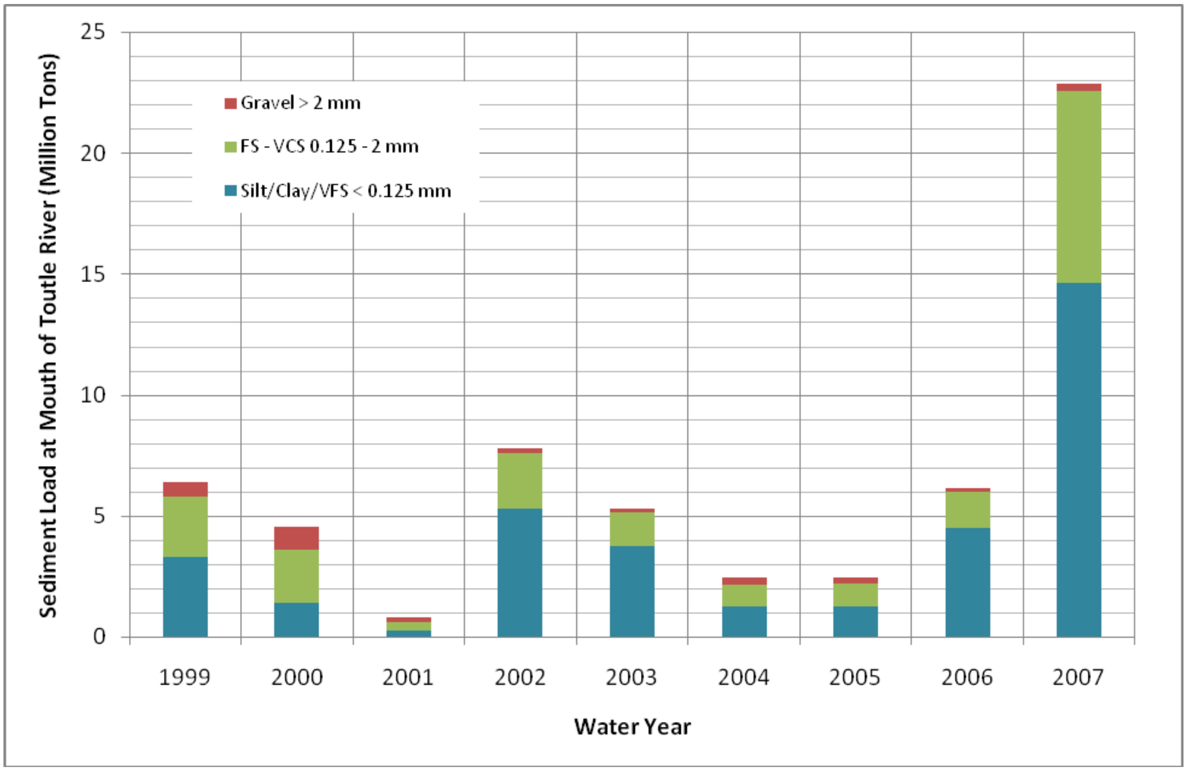


Figure 1.2 Annual Sediment Load by Grain Class at Mouth of Toutle River

Forecasting to 2035 of the sediment load at the mouth of the Toutle River was conducted using randomly selected series of the nine predicted annual sediment budgets, incorporating a Monte-Carlo Type analysis. The cumulative sediment load at the mouth of the Toutle River by 2035 was estimated to be between 94 and 338 million tons. The sequence of years generating the maximum, minimum, 5%, 50%, and 95% exceedance values of the sediment load by 2035 are presented in Table 1.1. and graphically in Figure 1.3. The Monte-Carlo procedure is discussed in detail in the Toutle/Cowlitz Sediment Budget report.

Analysis of measures to mitigate sedimentation in the lower Cowlitz will be conducted using the sediment budget and forecasting methods. Evaluation of the measures analysis results will be reviewed relative to the forecast of cumulative sediment load at the mouth of the Toutle River by 2035.

Table 1.1 Forecast of Cumulative Sediment Load at Mouth of Toutle River for Existing Conditions

Sequence	Maximum		5% Exceedance		50% Exceedance		95% Exceedance		100% Exceedance	
Forecast Year	WY	(M Tons)	WY	(M Tons)	WY	(M Tons)	WY	(M Tons)	WY	(M Tons)
2008	2007	22.8	2001	0.7	2001	0.7	2001	0.7	2003	5.3
2009	2001	23.6	2007	23.6	2002	8.4	2003	6.0	2004	7.8
2010	2007	46.4	2006	29.7	2007	31.2	1999	12.3	2004	10.2
2011	1999	52.7	2007	52.5	2002	38.9	1999	18.6	2001	10.9
2012	2004	55.1	1999	58.8	2000	43.4	1999	24.8	1999	17.2
2013	2001	55.8	2002	66.4	2003	48.7	2004	27.3	2004	19.7
2014	2003	61.1	2002	74.1	1999	55.0	2000	31.8	2004	22.1
2015	2000	65.7	1999	80.3	2005	57.4	1999	38.1	2005	24.6
2016	2007	88.5	2005	82.8	2007	80.3	2006	44.2	2002	32.3
2017	2007	111.3	2006	88.9	2003	85.6	2002	51.9	1999	38.5
2018	2007	134.2	2001	89.7	2001	86.3	2006	58.0	2000	43.1
2019	2007	157.0	2000	94.2	2003	91.6	2000	62.5	2002	50.7
2020	2006	163.1	2002	101.9	2004	94.1	2003	67.8	2005	53.2
2021	2000	167.7	2003	107.2	2002	101.7	2001	68.6	2006	59.3
2022	2003	173.0	2007	130.0	2000	106.2	2003	73.9	2005	61.8
2023	2003	178.3	2006	136.1	1999	112.5	2000	78.4	2001	62.5
2024	2007	201.1	2005	138.6	2005	115.0	2004	80.9	1999	68.8
2025	2007	223.9	2000	143.1	2007	137.8	2003	86.2	2004	71.2
2026	1999	230.2	2000	147.7	2004	140.3	2004	88.6	2005	73.7
2027	2001	230.9	2007	170.5	2001	141.0	2002	96.3	2001	74.4
2028	2003	236.2	2005	173.0	2003	146.3	1999	102.5	2004	76.9
2029	2000	240.8	2007	195.8	2004	148.8	2003	107.8	2001	77.6
2030	2007	263.6	2001	196.5	2006	154.9	2003	113.1	2004	80.1
2031	2001	264.3	1999	202.8	2006	161.0	2005	115.6	2004	82.5
2032	2007	287.1	2007	225.6	2002	168.7	2001	116.3	2001	83.3
2033	2007	310.0	2004	228.1	2006	174.8	2003	121.6	2001	84.0
2034	2007	332.8	2000	232.6	2005	177.2	2005	124.1	2003	89.3
2035	**2006**	**338.9**	**1999**	**238.9**	**2004**	**179.7**	**2002**	**131.8**	**2000**	**93.8**

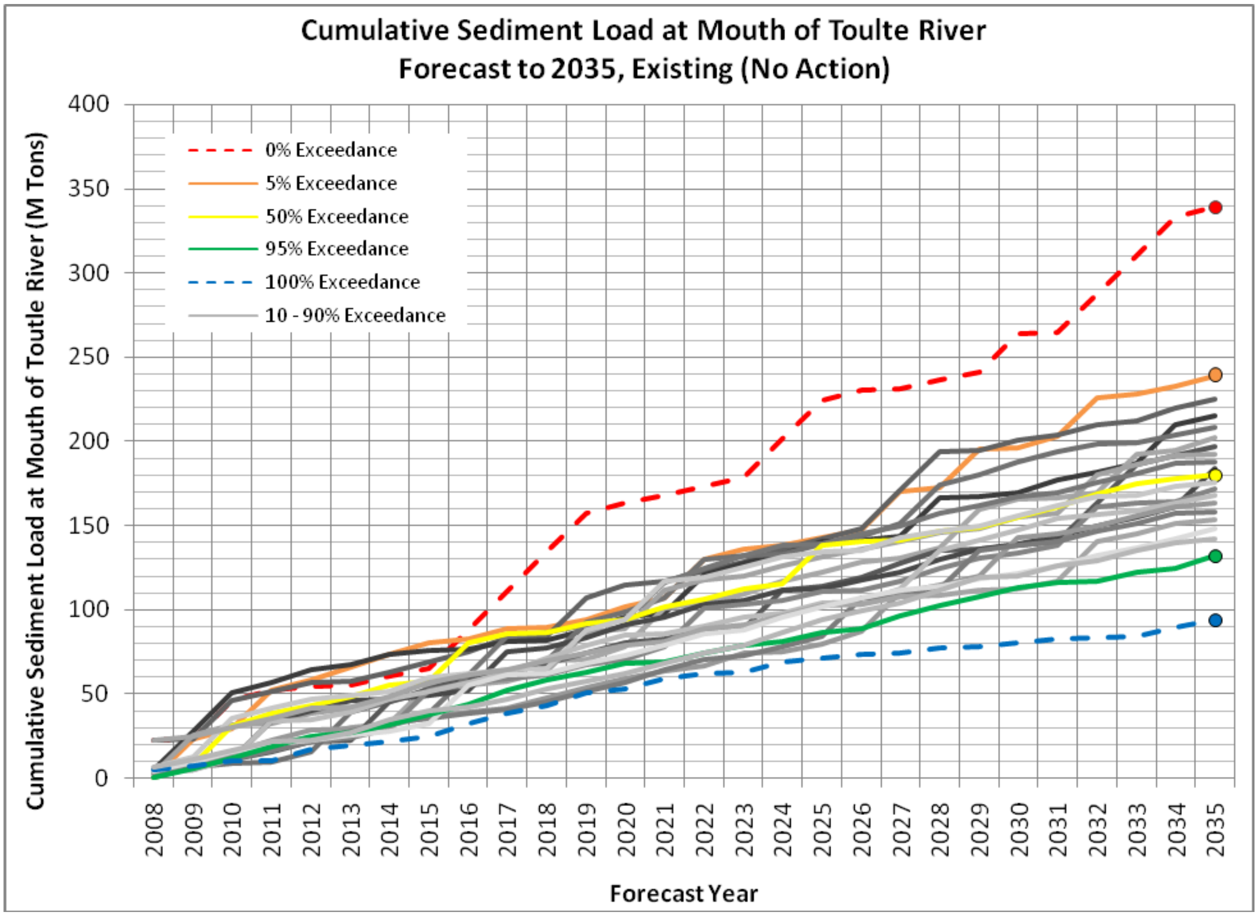


Figure 1.3 Forecast of Cumulative Sediment Load at Mouth of Toutle River by 2035 for Existing Conditions

2.0 Measures

Level 1 screening using the sediment budget was conducted for three of the proposed measures: the raised SRS, grade control structures upstream of the SRS, and a sump at LT1. The raised SRS and grade control structures were both analyzed as standalone measures and in combination with the LT1 Sump, a total of four separate analyses: 1) Raised SRS, 2) Raised SRS + LT1, 3) Grade Control Structures, and 4) Grade Control Structures + LT1.

2.1 Raised SRS

Operation of a raised SRS, based on estimates of planning, design, and construction time, would likely commence at the beginning of water year 2015 and provide a storage capacity at zero slope of approximately 500 MCY or 641 Million Tons (M Tons). The proposed outlet works of the raised SRS structure would include tiered outlet pipes with gates that would operated similar to the original SRS.

Operation of the gates would keep the pool behind the structure as small as possible to ensure that larger material is not passed through the outlet.

The trap efficiency of the raised SRS was calculated by grain size using Equation 2.1. The original SRS was operational below its spillway between 1988 and 1998. Estimated trap efficiency of the original SRS was determined using the sediment budget between 1988 and 1998. Calculated trap efficiency of the raised SRS is compared to sediment budget calculations in Figure 2.1.

$TE_i = 1 - e^{-X\omega i/hV}$ Equation 2.1

Where:

TE_i = trap efficiency

i = grain size

X = distance of travel

ω_i = fall velocity of given grain size

h = average depth

V = average velocity

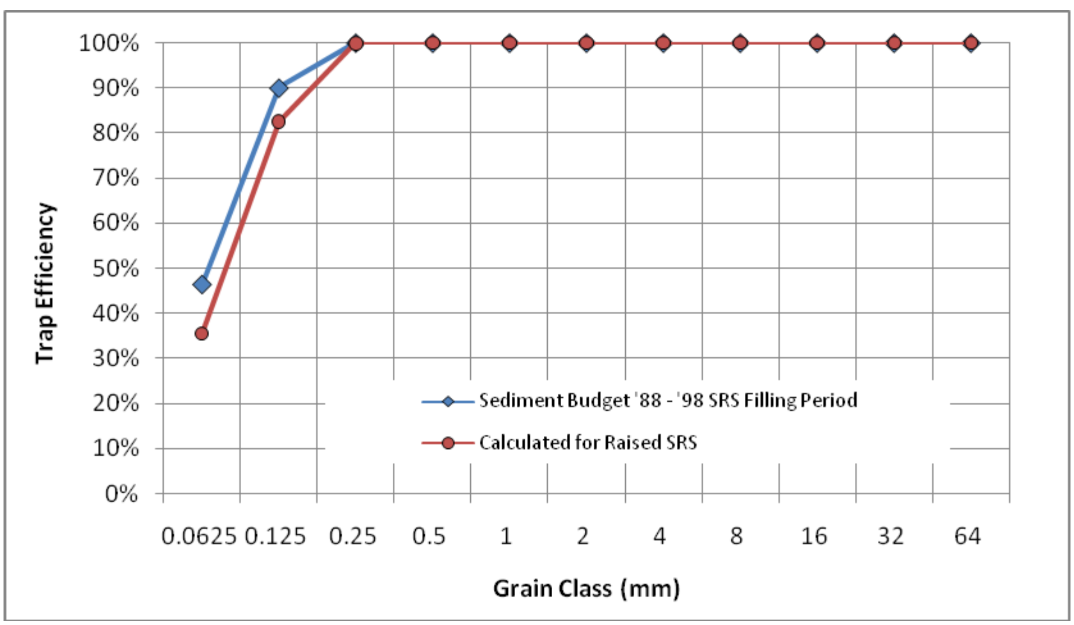


Figure 2.1 Raised SRS Trap Efficiency

2.2 Grade Control Structures

This measure includes layered construction of 10 sets of grade control structures on the sediment plain upstream of the SRS. Operation of this measure would likely begin at the start of water year 2012. Each structure extends across the sediment plain, is approximately 6 feet in total height, and has a spillway height of 3 feet. Construction of one set of 10 structures would provide a maximum of 8 MCY or 10.3 M Tons of storage. As the structures fill with sediment an additional set will sequentially be built on top of the deposited sediment. Construction of a new set of structures would be conducted when deposition is between 6 and 8 MCY or 7.7 and 10.3 M Tons. Figure 2.2 shows the layout of one set of grade control structures.

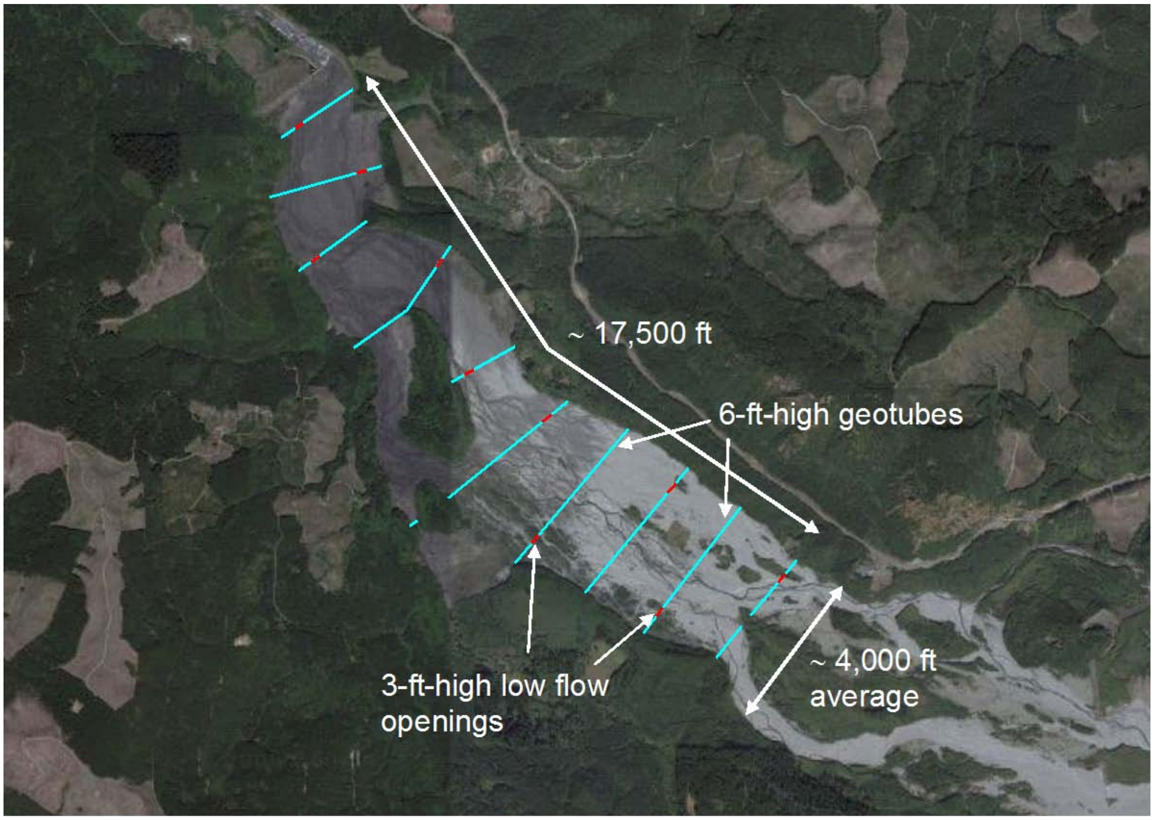


Figure 2.2 Example Layout of One Set of Grade Control Structures

Trap efficiency by grain class for the grade control structures was calculated for two conditions including 1) before sediment deposition reaches the spillway and storage capacity is available, and 2) after the structures have filled to the spillway and before an additional set can be built. Trap efficiency was calculated for both conditions using Equation 2.1 and is shown graphically in Figure 2.3.

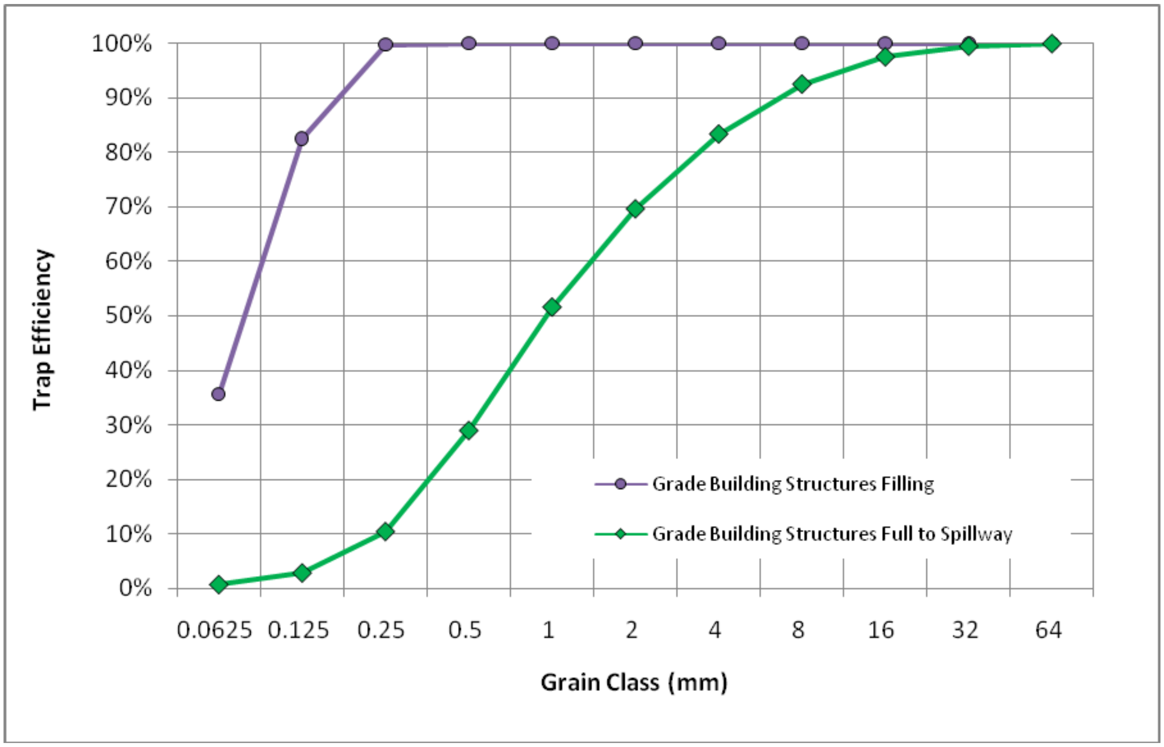


Figure 2.3 Grade Control Structures Trap Efficiency

2.3 LT1 Sump

The LT1 sump measure includes annual in-channel excavation with a surface area of approximately 2.3 million square feet, a maximum depth of 24 feet, and a maximum annual capacity of 2 MCY or 2.5 M Tons. Sump excavation will be conducted annually during the July – September in-water work period. This measure includes an on-site disposal site with a maximum capacity of approximately 20 MCY or 25 M Tons. Stabilization of channel banks in the vicinity of LT1 will be included with the measure. Operation of the LT1 sump would commence at the beginning of water year 2011.

The ability of an in-channel sump to trap sediment is dependent upon the mode by which sediment is transported. Sediment moving as bedload can readily be trapped with a given efficiency however sediment moving in suspension will likely pass over top of the sump and continue downstream. A HEC-

RAS hydraulic model of the LT1 site was utilized to calculate hydraulic conditions for a range of discharges. Discharges correspond to a 15 point average annual flow duration curve generated from the USGS Toutle at Tower Road daily gage data. Hydraulic analyses of the LT1 site assume that excavation of the sump will not drastically alter the one-dimensional hydraulic conditions. Output from the hydraulic model was utilized to determine the duration that a given grain size is moving in suspension or by bedload for an average water year. Shear velocity divided by fall velocity is plotted versus dimensionless shear stress divided by critical dimensionless shear stress for each discharge and grain size in Figure 2.4. Results of the hydraulic analysis were combined with the trap efficiency equation and weighted by the flow duration curve to develop annual trap efficiency by grain size for the LT1 sump, see Table 2.1. Material < 0.5 mm is in suspension for the entire year and will not be trapped by the sump.

Findings of the analysis that no deposition of medium sand and finer (<0.5 mm) would occur at an LT-1 sump, varies from the material observed in the existing LT-1 disposal pile. A recent sample of the eroding LT-1 bank (Biedenharn Group, Sediment Budget) contained greater than 50% medium sand and finer; however, sediment concentrations were higher immediately after eruption than the current conditions resulting in generally higher deposition rates. Presence of these materials in the dredge disposal pile does warrant additional investigation of potential sump performance. The analysis does indicate that the proposed sump would be effective at trapping all classes of gravels. If deposition of these materials in the quantities delivered to the Cowlitz is determined to be problematic, reactivation of LT-1 may present a viable flood protection measure, specifically for communities located closer to the Toutle.

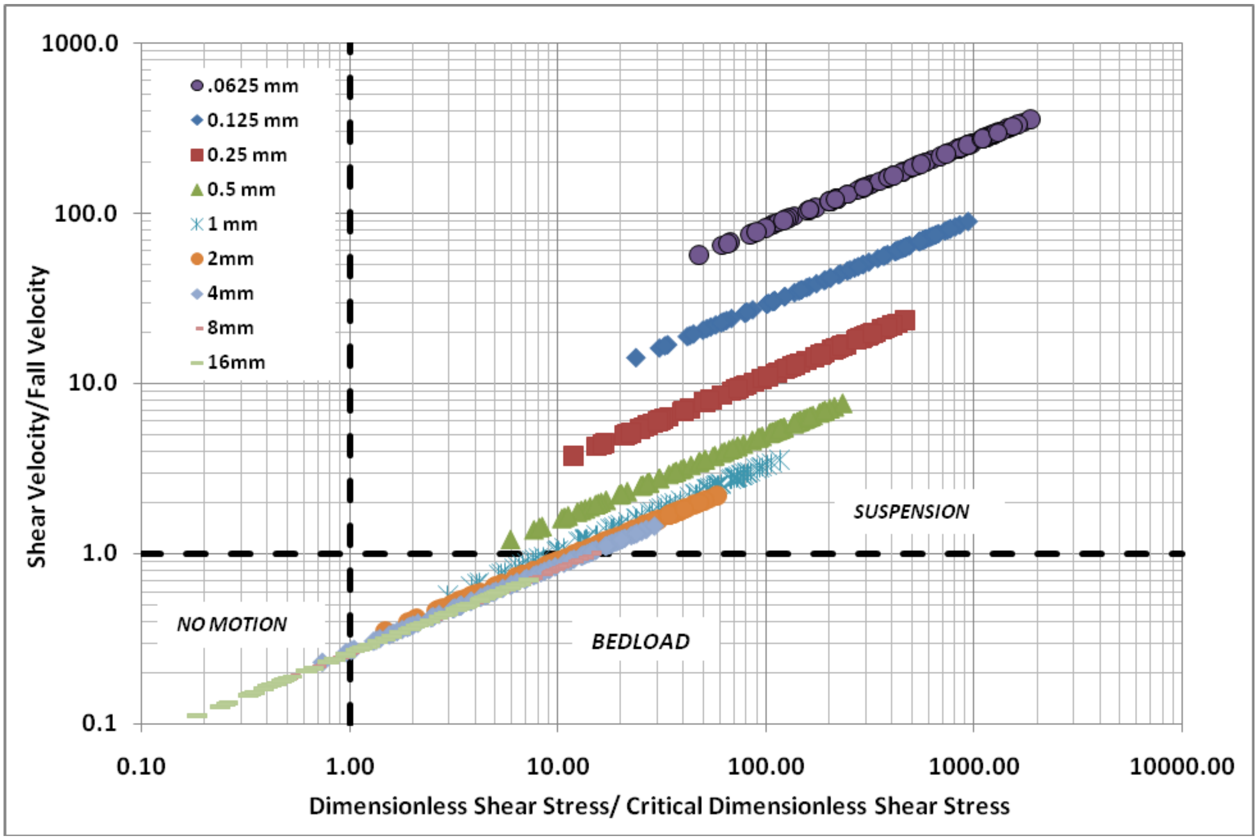


Figure 2.4 The shear stress ratio and shear velocity/fall velocity ratio combine to portray zones of motion, no motion, bed load and suspended load at LT1.

Table 2.1 LT1 Trap Efficiency

				Grain Class	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
				Grain Size < (mm)	**0.0625**	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**	**63**
				ω_i Fall Velocity of particle size i (mm/sec) [A]	0.67	2.66	10.1	31.3	66.4	109	164	237	338	479	678
				ω_i Fall Velocity of particle size i (ft/sec) [A]	0.002	0.009	0.03	0.10	0.22	0.36	0.54	0.78	1.11	1.57	2.22
				X Distance Across Sump (ft)	**3,000**										
Discharge [B]	**Depth (h)** [C]	**Velocity (V)** [C]	**Duration** [B]		**Trap Efficiency %**										
(cfs)	**(ft)**	**(ft/s)**	**(days/yr)**	**%**	**Suspension** [D] **TE = 0**				**Bedload TE = $1-e^{-X\omega i/hV}$**						
291	1.1	2.0	12.0	3.3	0	0	0	0	100	100	100	100	100	100	100
400	1.4	2.2	44.6	12.2	0	0	0	0	100	100	100	100	100	100	100
551	1.7	2.5	35.5	9.7	0	0	0	0	100	100	100	100	100	100	100
759	1.9	2.8	32.1	8.8	0	0	0	0	0	100	100	100	100	100	100
1044	2.3	3.1	34.3	9.4	0	0	0	0	0	100	100	100	100	100	100
1437	2.7	3.2	42.5	11.6	0	0	0	0	0	100	100	100	100	100	100
1978	3.2	3.3	60.2	16.5	0	0	0	0	0	100	100	100	100	100	100
2723	3.7	3.5	45.3	12.4	0	0	0	0	0	0	100	100	100	100	100
3748	4.4	3.8	27.1	7.4	0	0	0	0	0	0	100	100	100	100	100
5159	5.1	4.2	16.2	4.4	0	0	0	0	0	0	100	100	100	100	100
7101	5.9	4.6	8.1	2.2	0	0	0	0	0	0	0	100	100	100	100
9774	7.0	4.8	4.3	1.2	0	0	0	0	0	0	0	100	100	100	100
13452	8.2	4.9	1.3	0.4	0	0	0	0	0	0	0	100	100	100	100
18516	9.3	5.1	1.2	0.3	0	0	0	0	0	0	0	100	100	100	100
25486	10.4	5.6	0.2	0.1	0	0	0	0	0	0	0	100	100	100	100
Average Annual Trap Efficiency Weighted by Flow Duration					**0.0**	**0.0**	**0.0**	**0.0**	**25.2**	**71.6**	**95.8**	**100**	**100**	**100**	**100**

[A] Erosion and Sedimentation, Julien 1994, Table 5.4, Fall Velocity @ 10 deg C

[B] Annual flow duration curve developed from Toutle at Tower Road USGS daily discharge data 1999 - 2007

[C] Average hydraulic conditions obtained from HEC-RAS model cross sections 18820 - 14257

[D] Suspension or bedload determined from comparison of shear/fall velocity and dimensionless shear/critical shear

3.0 Analysis of Measures

Analysis of each individual measure was conducted by modifying the 1999 – 2007 annual sediment budgets. Results of the modified annual sediment budgets were then used to forecast of the cumulative sediment load to 2035 at the mouth of the Toutle River.

An annual budget for each measure was formulated by modifying the existing budget. Therefore, each water year has a total of six sediment budgets including: 1) raised SRS, 2) grade control structures, 3) LT1 sump, 4) raised SRS and LT1 sump, and 5) grade control structures and LT1 sump.

Each measure was incorporated into annual sediment budgets by applying the corresponding trap efficiency to the incoming load while also ensuring that the capacity of each measure was not exceeded. An example of the six sediment budgets is presented in Tables 3.1 through 3.6 for water year 2007. The remaining budgets for water years 1999 – 2006 are provided in the enclosed digital files. Tables 3.7 – 3.13 provide summary output of the annual sediment budgets by grain class including debris avalanche erosion, deposition behind the SRS, output from the SRS, and sediment load at the mouth of the Toutle River.

Table 3.1 Toutle/Cowlitz Sediment Budget Water Year 2007, Existing Conditions (No Action)

Toutle/Cowlitz River Sediment Budget
From Debris Avalanche to Columbia River
WY 2007

				Silts	Sand					Gravel					Cobble	
	Description	Data Source/Notes	Total M Tons	CM 0.0625 Ton	VFS 0.125 Ton	FS 0.25 Ton	MS 0.5 Ton	CS 1 Ton	VCS 2 Ton	VFG 4 Ton	FG 8 Ton	MG 16 Ton	CG 32 Ton	VCG 63 Ton	SC 128 Ton	LC 256 Ton
North Fork Toutle River: Debris Avalanche to SRS																
Debris Avalanche Erosion	Coldwater Creek	1999-2007 Surface Comparison Pro-rated by: Tower SS-South Fork SS + SRS Deposition	1,678,475	520,327	318,910	302,126	201,417	117,493	67,139	25,177	25,177	33,570	33,570	33,570	0	0
	Castle Creek		4,577,135	1,418,912	869,656	823,884	549,256	320,399	183,085	68,657	68,657	91,543	91,543	91,543	0	0
	Loowit		9,030,120	2,799,337	1,715,723	1,625,422	1,083,614	632,108	361,205	135,452	135,452	180,602	180,602	180,602	0	0
	A - Debris Avalanche to Elk Rock		5,552,558	1,721,293	1,054,986	999,460	666,307	388,679	222,102	83,288	83,288	111,051	111,051	111,051	0	0
	B - Elk Rock to N1		5,359,368	1,661,404	1,018,280	964,686	643,124	375,156	214,375	80,391	80,391	107,187	107,187	107,187	0	0
SRS Deposition	C - Sediment Plane	2006-2007 Surface Comparison	(6,156,997)	(25,736)	(195,977)	(1,185,776)	(1,519,978)	(719,815)	(337,403)	(189,574)	(287,593)	(474,828)	(564,905)	(655,412)	0	0
	D - Sediment Plane		(2,151,180)	(27,341)	(192,746)	(810,608)	(561,824)	(207,782)	(91,597)	(50,574)	(80,562)	(80,691)	(37,775)	(9,680)	0	0
	E - Sediment Plane		(480,059)	(118,051)	(124,124)	(214,514)	(18,890)	(2,021)	(1,349)	(677)	(312)	(120)	0	0	0	0
Sources	Total Erosion	Sum of Debris Avalanche Erosion	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953	0	0
Sinks	Total Deposition Behind SRS	Sum of Sediment Plane Deposition	(8,788,236)	(171,129)	(512,847)	(2,210,898)	(2,100,692)	(929,618)	(430,350)	(240,825)	(368,467)	(555,638)	(602,679)	(665,093)	0	0
Output from SRS	Output to North Fork Toutle River	Erosion - Deposition	17,409,420	7,950,144	4,464,708	2,504,680	1,043,027	904,218	617,557	152,140	24,498	(31,685)	(78,726)	(141,140)	0	0
North Fork Toutle River: SRS to Toutle River																
Input	Output from SRS		17,660,971	7,950,144	4,464,708	2,504,680	1,043,027	904,218	617,557	152,140	24,498	0	0	0	0	0
Sources	Bank Erosion North Fork Toutle	Est. & pro-rated from 99-06 Aerial Photos	94,617	3,270	4,495	10,951	20,083	17,117	12,395	7,115	4,930	6,274	3,634	4,353	0	
	Green River	Estimate from USGS Gage Data + 18% Unmeasured	158,366	69,401	21,569	28,007	24,571	11,298	3,519							
Sinks																
Output	Output to Toutle River		17,913,954	8,022,815	4,490,772	2,543,638	1,087,681	932,633	633,471	159,255	29,428	6,274	3,634	4,353	0	0
South Fork Toutle River: Upstream of USGS Gage																
Input	Upstream Source = Gage - Bank Erosion	Upstream Source Data Unavaliable	4,528,203	1,266,759	823,619	1,346,629	981,867	179,346	(9,485)	(20,216)	(12,623)	(12,841)	(9,802)	(5,052)	0	0
Sources	Bank Erosion South Fork	Est. & pro-rated from 99-06 Aerial Photos	212,148	7,169	13,961	25,744	29,009	38,934	36,797	20,216	12,623	12,841	9,802	5,052	0	
Sinks																
Output	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,740,351	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	0	0	0	0	0	0	0
South Fork Toutle River: Downstream of USGS Gage																
Input	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Sources																
Sinks																
Output	Output to Toutle River		4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Toutle River: Confluence of North Fork and South Fork to USGS Gage at Tower Road																
Input	Output from North Fork and South Fork		22,714,838	9,296,743	5,328,352	3,916,011	2,098,558	1,150,913	660,784	179,470	42,051	19,115	13,436	9,406	0	0
Sources	Toutle Bank Erosion Above Tower	Est. & pro-rated from 99-06 Aerial Photos	20,822	1,814	2,560	3,173	3,236	2,843	2,059	1,396	1,210	760	1,091	679	0	
Sinks																
Output at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	22,735,660	9,298,557	5,330,912	3,919,184	2,101,794	1,153,756	662,843	180,867	43,261	19,874	14,527	10,084	0	0
Toutle River: USGS Gage at Tower Road to Cowlitz River																
Input at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	22,735,660	9,298,557	5,330,912	3,919,184	2,101,794	1,153,756	662,843	180,867	43,261	19,874	14,527	10,084	0	0
Sources	Toutle Bank Erosion Below Tower	Est. & pro-rated from 99-06 Aerial Photos	92,034	3,922	5,761	12,921	27,212	15,354	7,279	4,290	3,701	4,705	6,413	476	0	
Sinks																
Output	Output to Cowlitz River		22,827,694	9,302,479	5,336,673	3,932,105	2,129,006	1,169,110	670,123	185,156	46,962	24,580	20,940	10,561	0	0
Cowlitz River: Toutle River to Columbia River																
Input	Input from Toutle River		22,827,694	9,302,479	5,336,673	3,932,105	2,129,006	1,169,110	670,123	185,156	46,962	24,580	20,940	10,561	0	0
	Input from Upper Cowlitz		0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sources			0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sinks	Cowlitz River Deposition/Erosion	Hydro-Survey Comparisons	(1,526,837)	(37,670)	(126,425)	(523,045)	(310,403)	(210,151)	(50,049)	(10,209)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	
Output	Output to Columbia River		21,300,858	9,264,809	5,210,248	3,409,060	1,818,603	958,959	620,074	174,947	14,849	(6,281)	(30,642)	(113,227)	(20,543)	0

(Note: Negative values indicate deposition or sinks, Positive values indicate erosion or sources)

Table 3.2 Toutle/Cowlitz Sediment Budget Water Year 2007, Raised SRS

Toutle/Cowlitz River Sediment Budget RAISED SRS MEASURE
From Debris Avalanche to Columbia River
WY 2007

				Silts	Sand					Gravel					Cobble	
				CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG	SC	LC
			Total	0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63	128	256
Description		Data Source/Notes	M Tons	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton
North Fork Toutle River: Debris Avalanche to SRS																
Debris Avalanche Erosion	Coldwater Creek	1999-2007 Surface Comparison Pro-rated by: Tower SS-South Fork SS + SRS Deposition	1,678,475	520,327	318,910	302,126	201,417	117,493	67,139	25,177	25,177	33,570	33,570	33,570	0	0
	Castle Creek		4,577,135	1,418,912	869,656	823,884	549,256	320,399	183,085	68,657	68,657	91,543	91,543	91,543	0	0
	Loowit		9,030,120	2,799,337	1,715,723	1,625,422	1,083,614	632,108	361,205	135,452	135,452	180,602	180,602	180,602	0	0
	A - Debris Avalanche to Elk Rock		5,552,558	1,721,293	1,054,986	999,460	666,307	388,679	222,102	83,288	83,288	111,051	111,051	111,051	0	0
	B - Elk Rock to N1		5,359,368	1,661,404	1,018,280	964,686	643,124	375,156	214,375	80,391	80,391	107,187	107,187	107,187	0	0
Raised SRS Deposition		**Raised SRS Trap Efficiency**	**77%**	**36%**	**83%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**0%**	**0%**
		Deposition = Erosion * Trap Efficiency	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)	**0**	**0**
Sources	Total Erosion	Sum of Debris Avalanche Erosion	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953	0	0
Sinks	Total Deposition Behind Raised SRS	Sum of Sediment Plane Deposition	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)	0	0
Output from SRS	Output to North Fork Toutle River	Erosion - Deposition	6,107,512	5,232,318	868,951	6,242	0	0	0	0	0	0	0	0	0	0
North Fork Toutle River: SRS to Toutle River																
Input	Output from SRS		6,107,512	5,232,318	868,951	6,242	0	0	0	0	0	0	0	0	0	0
Sources	Bank Erosion North Fork Toutle	Est. & pro-rated from 99-06 Aerial Photos	94,617	3,270	4,495	10,951	20,083	17,117	12,395	7,115	4,930	6,274	3,634	4,353	0	
	Green River	Estimate from USGS Gage Data + 18% Unmeasured	158,366	69,401	21,569	28,007	24,571	11,298	3,519							
Sinks																
Output	Output to Toutle River		6,360,495	5,304,989	895,016	45,200	44,655	28,415	15,915	7,115	4,930	6,274	3,634	4,353	0	0
South Fork Toutle River: Upstream of USGS Gage																
Input	Upstream Source = Gage - Bank Erosion	Upstream Source Data Unavaliable	4,528,203	1,266,759	823,619	1,346,629	981,867	179,346	(9,485)	(20,216)	(12,623)	(12,841)	(9,802)	(5,052)	0	0
Sources	Bank Erosion South Fork	Est. & pro-rated from 99-06 Aerial Photos	212,148	7,169	13,961	25,744	29,009	38,934	36,797	20,216	12,623	12,841	9,802	5,052	0	
Sinks																
Output	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,740,351	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	0	0	0	0	0	0	0
South Fork Toutle River: Downstream of USGS Gage																
Input	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Sources																
Sinks																
Output	Output to Toutle River		4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Toutle River: Confluence of North Fork and South Fork to USGS Gage at Tower Road																
Input	Output from North Fork and South Fork		11,161,379	6,578,917	1,732,596	1,417,573	1,055,531	246,695	43,227	27,331	17,553	19,115	13,436	9,406	0	0
Sources	Toutle Bank Erosion Above Tower	Est. & pro-rated from 99-06 Aerial Photos	20,822	1,814	2,560	3,173	3,236	2,843	2,059	1,396	1,210	760	1,091	679	0	
Sinks																
Output at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	11,182,201	6,580,731	1,735,155	1,420,747	1,058,767	249,539	45,287	28,727	18,763	19,874	14,527	10,084	0	0
Toutle River: USGS Gage at Tower Road to Cowlitz River																
Input at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	11,182,201	6,580,731	1,735,155	1,420,747	1,058,767	249,539	45,287	28,727	18,763	19,874	14,527	10,084	0	0
Sources	Toutle Bank Erosion Below Tower	Est. & pro-rated from 99-06 Aerial Photos	92,034	3,922	5,761	12,921	27,212	15,354	7,279	4,290	3,701	4,705	6,413	476	0	
Sinks																
Output	Output to Cowlitz River		11,274,235	6,584,653	1,740,916	1,433,668	1,085,979	264,892	52,566	33,017	22,464	24,580	20,940	10,561	0	0
Cowlitz River: Toutle River to Columbia River																
Input	Input from Toutle River		11,274,235	6,584,653	1,740,916	1,433,668	1,085,979	264,892	52,566	33,017	22,464	24,580	20,940	10,561	0	0
	Input from Upper Cowlitz		0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sources			0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sinks	Cowlitz River Deposition/Erosion	Hydro-Survey Comparisons	(1,526,837)	(37,670)	(126,425)	(523,045)	(310,403)	(210,151)	(50,049)	(10,209)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	
Output	Output to Columbia River		9,747,399	6,546,983	1,614,491	910,622	775,576	54,742	2,518	22,808	(9,649)	(6,281)	(30,642)	(113,227)	(20,543)	0

(Note: Negative values indicate deposition or sinks, Positive values indicate erosion or sources)

Table 3.3 Toutle/Cowlitz Sediment Budget Water Year 2007, Grade Control Structures

Toutle/Cowlitz River Sediment Budget GRADE BUILDING STRUCTURES ABOVE SRS
From Debris Avalanche to Columbia River
WY 2007

				Silts	Sand					Gravel					Cobble	
				CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG	SC	LC
			Total	0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63	128	256
Description		Data Source/Notes	M Tons	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton
North Fork Toutle River: Debris Avalanche to SRS																
Debris Avalanche Erosion	Coldwater Creek	1999-2007 Surface Comparison Pro-rated by: Tower SS-South Fork SS + SRS Deposition	1,678,475	520,327	318,910	302,126	201,417	117,493	67,139	25,177	25,177	33,570	33,570	33,570	0	0
	Castle Creek		4,577,135	1,418,912	869,656	823,884	549,256	320,399	183,085	68,657	68,657	91,543	91,543	91,543	0	0
	Loowit		9,030,120	2,799,337	1,715,723	1,625,422	1,083,614	632,108	361,205	135,452	135,452	180,602	180,602	180,602	0	0
	A - Debris Avalanche to Elk Rock		5,552,558	1,721,293	1,054,986	999,460	666,307	388,679	222,102	83,288	83,288	111,051	111,051	111,051	0	0
	B - Elk Rock to N1		5,359,368	1,661,404	1,018,280	964,686	643,124	375,156	214,375	80,391	80,391	107,187	107,187	107,187	0	0
Grade Building Structures Above SRS		**Trap Efficiency - Operational (Filling to 10.3 MTons)**		**36%**	**83%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**0%**	**0%**
		Trap Efficiency - Non-Operational (Full)		1%	3%	10%	29%	52%	70%	83%	93%	98%	99%	100%	0%	0%
		Deposition = Erosion * Trap Efficiency (Filling)	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)		
		Deposition = Erosion * Trap Efficiency (Full)	(5,529,721)	(59,289)	(142,712)	(493,124)	(911,256)	(946,693)	(729,760)	(327,586)	(363,539)	(510,953)	(521,172)	(523,638)		
		Deposition (Filling)	51%	(1,481,136)	(2,106,437)	(2,414,426)	(1,611,751)	(940,188)	(537,250)	(201,469)	(201,469)	(268,625)	(268,625)	(268,625)		
		Deposition (Full)	49%	(28,892)	(69,545)	(240,304)	(444,065)	(461,334)	(355,620)	(159,636)	(177,157)	(248,993)	(253,973)	(255,174)		
Sources	Total Erosion	Sum of Debris Avalanche Erosion	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953	0	0
Sinks	Total Deposition Behind GBS	Sum of Deposition	(12,994,693)	(1,510,028)	(2,175,982)	(2,654,730)	(2,055,815)	(1,401,522)	(892,870)	(361,105)	(378,626)	(517,618)	(522,598)	(523,799)	0	0
Output from SRS	Output to North Fork Toutle River	Erosion - Deposition	13,202,963	6,611,245	2,801,573	2,060,848	1,087,903	432,314	155,036	31,860	14,339	6,335	1,355	154	0	0
North Fork Toutle River: SRS to Toutle River																
Input	Output from SRS		13,202,963	6,611,245	2,801,573	2,060,848	1,087,903	432,314	155,036	31,860	14,339	6,335	1,355	154	0	0
Sources	Bank Erosion North Fork Toutle	Est. & pro-rated from 99-06 Aerial Photos	94,617	3,270	4,495	10,951	20,083	17,117	12,395	7,115	4,930	6,274	3,634	4,353	0	
	Green River	Estimate from USGS Gage Data + 18% Unmeasured	158,366	69,401	21,569	28,007	24,571	11,298	3,519							
Sinks																
Output	Output to Toutle River		13,455,946	6,683,916	2,827,637	2,099,806	1,132,558	460,729	170,951	38,975	19,270	12,609	4,989	4,507	0	0
South Fork Toutle River: Upstream of USGS Gage																
Input	Upstream Source = Gage - Bank Erosion	Upstream Source Data Unavaliable	4,528,203	1,266,759	823,619	1,346,629	981,867	179,346	-9,485	-20,216	-12,623	-12,841	-9,802	-5,052	0	0
Sources	Bank Erosion South Fork	Est. & pro-rated from 99-06 Aerial Photos	212,148	7,169	13,961	25,744	29,009	38,934	36,797	20,216	12,623	12,841	9,802	5,052	0	
Sinks																
Output	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,740,351	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	0	0	0	0	0	0	0
South Fork Toutle River: Downstream of USGS Gage																
Input	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Sources																
Sinks																
Output	Output to Toutle River		4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Toutle River: Confluence of North Fork and South Fork to USGS Gage at Tower Road																
Input	Output from North Fork and South Fork		18,256,831	7,957,844	3,665,217	3,472,179	2,143,434	679,010	198,263	59,190	31,892	25,450	14,791	9,560	0	0
Sources	Toutle Bank Erosion Above Tower	Est. & pro-rated from 99-06 Aerial Photos	20,822	1,814	2,560	3,173	3,236	2,843	2,059	1,396	1,210	760	1,091	679	0	
Sinks																
Output at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	18,277,652	7,959,658	3,667,777	3,475,352	2,146,671	681,853	200,323	60,586	33,103	26,210	15,882	10,238	0	0
Toutle River: USGS Gage at Tower Road to Cowlitz River																
Input at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	18,277,652	7,959,658	3,667,777	3,475,352	2,146,671	681,853	200,323	60,586	33,103	26,210	15,882	10,238	0	0
Sources	Toutle Bank Erosion Below Tower	Est. & pro-rated from 99-06 Aerial Photos	92,034	3,922	5,761	12,921	27,212	15,354	7,279	4,290	3,701	4,705	6,413	476	0	
Sinks																
Output	Output to Cowlitz River		18,369,687	7,963,580	3,673,538	3,488,273	2,173,882	697,207	207,602	64,876	36,803	30,915	22,295	10,714	0	0
Cowlitz River: Toutle River to Columbia River																
Input	Input from Toutle River		18,369,687	7,963,580	3,673,538	3,488,273	2,173,882	697,207	207,602	64,876	36,803	30,915	22,295	10,714	0	0
	Input from Upper Cowlitz		0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sources			0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sinks	Cowlitz River Deposition/Erosion	Hydro-Survey Comparisons	(1,526,837)	(37,670)	(126,425)	(523,045)	(310,403)	(210,151)	(50,049)	(10,209)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	
Output	Output to Columbia River		16,842,850	7,925,910	3,547,113	2,965,228	1,863,480	487,056	157,554	54,667	4,691	55	(29,286)	(113,073)	(20,543)	0

(Note: Negative values indicate deposition or sinks, Positive values indicate erosion or sources)

Table 3.4 Toutle/Cowlitz Sediment Budget Water Year 2007, LT1 Sump

Toutle/Cowlitz River Sediment Budget LT1 SUMP
From Debris Avalanche to Columbia River
WY 2007

				Silts	Sand					Gravel					Cobble	
			Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG	SC	LC
				0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63	128	256
Description		Data Source/Notes	M Tons	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton
North Fork Toutle River: Debris Avalanche to SRS																
Debris Avalanche Erosion	Coldwater Creek	1999-2007 Surface Comparison Pro-rated by: Tower SS-South Fork SS + SRS Deposition	1,678,475	520,327	318,910	302,126	201,417	117,493	67,139	25,177	25,177	33,570	33,570	33,570	0	0
	Castle Creek		4,577,135	1,418,912	869,656	823,884	549,256	320,399	183,085	68,657	68,657	91,543	91,543	91,543	0	0
	Loowit		9,030,120	2,799,337	1,715,723	1,625,422	1,083,614	632,108	361,205	135,452	135,452	180,602	180,602	180,602	0	0
	A - Debris Avalanche to Elk Rock		5,552,558	1,721,293	1,054,986	999,460	666,307	388,679	222,102	83,288	83,288	111,051	111,051	111,051	0	0
	B - Elk Rock to N1		5,359,368	1,661,404	1,018,280	964,686	643,124	375,156	214,375	80,391	80,391	107,187	107,187	107,187	0	0
SRS Deposition	C - Sediment Plane	2006-2007 Surface Comparison	(6,156,997)	(25,736)	(195,977)	(1,185,776)	(1,519,978)	(719,815)	(337,403)	(189,574)	(287,593)	(474,828)	(564,905)	(655,412)	0	0
	D - Sediment Plane		(2,151,180)	(27,341)	(192,746)	(810,608)	(561,824)	(207,782)	(91,597)	(50,574)	(80,562)	(80,691)	(37,775)	(9,680)	0	0
	E - Sediment Plane		(480,059)	(118,051)	(124,124)	(214,514)	(18,890)	(2,021)	(1,349)	(677)	(312)	(120)	0	0	0	0
Sources	Total Erosion	Sum of Debris Avalanche Erosion	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953	0	0
Sinks	Total Deposition Behind SRS	Sum of Sediment Plane Deposition	(8,788,236)	(171,129)	(512,847)	(2,210,898)	(2,100,692)	(929,618)	(430,350)	(240,825)	(368,467)	(555,638)	(602,679)	(665,093)	0	0
Output from SRS	Output to North Fork Toutle River	Erosion - Deposition	17,409,420	7,950,144	4,464,708	2,504,680	1,043,027	904,218	617,557	152,140	24,498	(31,685)	(78,726)	(141,140)	0	0
North Fork Toutle River: SRS to Toutle River																
Input	Output from SRS		17,660,971	7,950,144	4,464,708	2,504,680	1,043,027	904,218	617,557	152,140	24,498	0	0	0	0	0
Sources	Bank Erosion North Fork Toutle	Est. & pro-rated from 99-06 Aerial Photos	94,617	3,270	4,495	10,951	20,083	17,117	12,395	7,115	4,930	6,274	3,634	4,353	0	
	Green River	Estimate from USGS Gage Data + 18% Unmeasured	158,366	69,401	21,569	28,007	24,571	11,298	3,519							
Sinks																
Output	Output to Toutle River		17,913,954	8,022,815	4,490,772	2,543,638	1,087,681	932,633	633,471	159,255	29,428	6,274	3,634	4,353	0	0
South Fork Toutle River: Upstream of USGS Gage																
Input	Upstream Source = Gage - Bank Erosion	Upstream Source Data Unavaliable	4,528,203	1,266,759	823,619	1,346,629	981,867	179,346	(9,485)	(20,216)	(12,623)	(12,841)	(9,802)	(5,052)	0	0
Sources	Bank Erosion South Fork	Est. & pro-rated from 99-06 Aerial Photos	212,148	7,169	13,961	25,744	29,009	38,934	36,797	20,216	12,623	12,841	9,802	5,052	0	
Sinks																
Output	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,740,351	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	0	0	0	0	0	0	0
South Fork Toutle River: Downstream of USGS Gage																
Input	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Sources																
Sinks																
Output	Output to Toutle River		4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Toutle River: Confluence of North Fork and South Fork to USGS Gage at Tower Road																
Input	Output from North Fork and South Fork		22,714,838	9,296,743	5,328,352	3,916,011	2,098,558	1,150,913	660,784	179,470	42,051	19,115	13,436	9,406	0	0
Sources	Toutle Bank Erosion Above Tower	Est. & pro-rated from 99-06 Aerial Photos	20,822	1,814	2,560	3,173	3,236	2,843	2,059	1,396	1,210	760	1,091	679	0	
Sinks																
Output at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	22,735,660	9,298,557	5,330,912	3,919,184	2,101,794	1,153,756	662,843	180,867	43,261	19,874	14,527	10,084	0	0
Toutle River: USGS Gage at Tower Road to Cowlitz River																
Input at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	22,735,660	9,298,557	5,330,912	3,919,184	2,101,794	1,153,756	662,843	180,867	43,261	19,874	14,527	10,084	0	0
Sources	Toutle Bank Erosion Below Tower	LT1 Bank Stab - Reduce Bank Erosion 50%	46,017	1,961	2,881	6,460	13,606	7,677	3,640	2,145	1,850	2,353	3,207	238	0	
Sinks	SUMP at LT1 Up to 2.5 M Tons Capacity	Trap Efficiency		0%	0%	0%	0%	25%	72%	96%	100%	100%	100%	100%		
		Sink = Inflowing load* Trap Efficiency	(1,040,921)	0	0	0	0	(293,184)	(476,957)	(175,386)	(45,111)	(22,227)	(17,733)	(10,322)		
Output	Output to Cowlitz River		21,740,756	9,300,518	5,333,792	3,925,645	2,115,400	868,249	189,526	7,625	0	0	0	0	0	0
Cowlitz River: Toutle River to Columbia River																
Input	Input from Toutle River		21,740,756	9,300,518	5,333,792	3,925,645	2,115,400	868,249	189,526	7,625	0	0	0	0	0	0
	Input from Upper Cowlitz		0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sources			0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sinks	Cowlitz River Deposition/Erosion	Hydro-Survey Comparisons	(1,526,837)	(37,670)	(126,425)	(523,045)	(310,403)	(210,151)	(50,049)	(10,209)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	
Output	Output to Columbia River		20,213,919	9,262,848	5,207,367	3,402,600	1,804,997	658,098	139,477	(2,584)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	0

(Note: Negative values indicate deposition or sinks, Positive values indicate erosion or sources)

Table 3.5 Toutle/Cowlitz Sediment Budget Water Year 2007, Raised SRS and LT1 Sump

Toutle/Cowlitz River Sediment Budget RAISED SRS MEASURE + LT1 SUMP
From Debris Avalanche to Columbia River
WY 2007

				Silts	Sand					Gravel					Cobble	
				CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG	SC	LC
			Total	0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63	128	256
Description		Data Source/Notes	M Tons	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton
North Fork Toutle River: Debris Avalanche to SRS																
Debris Avalanche Erosion	Coldwater Creek	1999-2007 Surface Comparison Pro-rated by: Tower SS-South Fork SS + SRS Deposition	1,678,475	520,327	318,910	302,126	201,417	117,493	67,139	25,177	25,177	33,570	33,570	33,570	0	0
	Castle Creek		4,577,135	1,418,912	869,656	823,884	549,256	320,399	183,085	68,657	68,657	91,543	91,543	91,543	0	0
	Loowit		9,030,120	2,799,337	1,715,723	1,625,422	1,083,614	632,108	361,205	135,452	135,452	180,602	180,602	180,602	0	0
	A - Debris Avalanche to Elk Rock		5,552,558	1,721,293	1,054,986	999,460	666,307	388,679	222,102	83,288	83,288	111,051	111,051	111,051	0	0
	B - Elk Rock to N1		5,359,368	1,661,404	1,018,280	964,686	643,124	375,156	214,375	80,391	80,391	107,187	107,187	107,187	0	0
Raised SRS Deposition		**Raised SRS Trap Efficiency**	**77%**	**36%**	**83%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**100%**	**0%**	**0%**
		Deposition = Erosion * Trap Efficiency	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)	**0**	**0**
Sources	Total Erosion	Sum of Debris Avalanche Erosion	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953	0	0
Sinks	Total Deposition Behind Raised SRS	Sum of Sediment Plane Deposition	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)	0	0
Output from SRS	Output to North Fork Toutle River	Erosion - Deposition	6,107,512	5,232,318	868,951	6,242	0	0	0	0	0	0	0	0	0	0
North Fork Toutle River: SRS to Toutle River																
Input	Output from SRS		6,107,512	5,232,318	868,951	6,242	0	0	0	0	0	0	0	0	0	0
Sources	Bank Erosion North Fork Toutle	Est. & pro-rated from 99-06 Aerial Photos	94,617	3,270	4,495	10,951	20,083	17,117	12,395	7,115	4,930	6,274	3,634	4,353	0	
	Green River	Estimate from USGS Gage Data + 18% Unmeasured	158,366	69,401	21,569	28,007	24,571	11,298	3,519							
Sinks																
Output	Output to Toutle River		6,360,495	5,304,989	895,016	45,200	44,655	28,415	15,915	7,115	4,930	6,274	3,634	4,353	0	0
South Fork Toutle River: Upstream of USGS Gage																
Input	Upstream Source = Gage - Bank Erosion	Upstream Source Data Unavaliable	4,528,203	1,266,759	823,619	1,346,629	981,867	179,346	(9,485)	(20,216)	(12,623)	(12,841)	(9,802)	(5,052)	0	0
Sources	Bank Erosion South Fork	Est. & pro-rated from 99-06 Aerial Photos	212,148	7,169	13,961	25,744	29,009	38,934	36,797	20,216	12,623	12,841	9,802	5,052	0	
Sinks																
Output	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,740,351	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	0	0	0	0	0	0	0
South Fork Toutle River: Downstream of USGS Gage																
Input	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Sources																
Sinks																
Output	Output to Toutle River		4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Toutle River: Confluence of North Fork and South Fork to USGS Gage at Tower Road																
Input	Output from North Fork and South Fork		11,161,379	6,578,917	1,732,596	1,417,573	1,055,531	246,695	43,227	27,331	17,553	19,115	13,436	9,406	0	0
Sources	Toutle Bank Erosion Above Tower	Est. & pro-rated from 99-06 Aerial Photos	20,822	1,814	2,560	3,173	3,236	2,843	2,059	1,396	1,210	760	1,091	679	0	
Sinks																
Output at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	11,182,201	6,580,731	1,735,155	1,420,747	1,058,767	249,539	45,287	28,727	18,763	19,874	14,527	10,084	0	0
Toutle River: USGS Gage at Tower Road to Cowlitz River																
Input at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	11,182,201	6,580,731	1,735,155	1,420,747	1,058,767	249,539	45,287	28,727	18,763	19,874	14,527	10,084	0	0
Sources	Toutle Bank Erosion Below Tower	LT1 Bank Stab - Reduce Bank Erosion 50%	46,017	1,961	2,881	6,460	13,606	7,677	3,640	2,145	1,850	2,353	3,207	238	0	
Sinks	SUMP at LT1 Up to 2.5 M Tons Capacity	Trap Efficiency		0%	0%	0%	0%	25%	72%	96%	100%	100%	100%	100%		
		Sink = Inflowing load* Trap Efficiency	(200,425)	0	0	0	0	(64,930)	(35,013)	(29,585)	(20,614)	(22,227)	(17,733)	(10,322)		
Output	Output to Cowlitz River		11,027,793	6,582,692	1,738,036	1,427,207	1,072,373	192,286	13,913	1,286	0	0	0	0	0	0
Cowlitz River: Toutle River to Columbia River																
Input	Input from Toutle River		11,027,793	6,582,692	1,738,036	1,427,207	1,072,373	192,286	13,913	1,286	0	0	0	0	0	0
	Input from Upper Cowlitz		0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sources			0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sinks	Cowlitz River Deposition/Erosion	Hydro-Survey Comparisons	(1,526,837)	(37,670)	(126,425)	(523,045)	(310,403)	(210,151)	(50,049)	(10,209)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	
Output	Output to Columbia River		9,500,957	6,545,022	1,611,611	904,162	761,970	(17,865)	(36,136)	(8,923)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	0

(Note: Negative values indicate deposition or sinks, Positive values indicate erosion or sources)

Table 3.6 Toutle/Cowlitz Sediment Budget Water Year 2007, Grade Control Structures and LT1 Sump

Toutle/Cowlitz River Sediment Budget GRADE BUILDING STRUCTURES ABOVE SRS + LT1 SUMP
From Debris Avalanche to Columbia River
WY 2007

				Silts	Sand					Gravel					Cobble	
				CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG	SC	LC
			Total	0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63	128	256
Description		Data Source/Notes	M Tons	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton	Ton
North Fork Toutle River: Debris Avalanche to SRS																
Debris Avalanche Erosion	Coldwater Creek	1999-2007 Surface Comparison Pro-rated by: Tower SS-South Fork SS + SRS Deposition	1,678,475	520,327	318,910	302,126	201,417	117,493	67,139	25,177	25,177	33,570	33,570	33,570	0	0
	Castle Creek		4,577,135	1,418,912	869,656	823,884	549,256	320,399	183,085	68,657	68,657	91,543	91,543	91,543	0	0
	Loowit		9,030,120	2,799,337	1,715,723	1,625,422	1,083,614	632,108	361,205	135,452	135,452	180,602	180,602	180,602	0	0
	A - Debris Avalanche to Elk Rock		5,552,558	1,721,293	1,054,986	999,460	666,307	388,679	222,102	83,288	83,288	111,051	111,051	111,051	0	0
	B - Elk Rock to N1		5,359,368	1,661,404	1,018,280	964,686	643,124	375,156	214,375	80,391	80,391	107,187	107,187	107,187	0	0
Grade Building Structures Above SRS		Trap Efficiency - Operational (Filling to 10.3 MTons)		36%	83%	100%	100%	100%	100%	100%	100%	100%	100%	100%	0%	0%
		Trap Efficiency - Non-Operational (Full)		1%	3%	10%	29%	52%	70%	83%	93%	98%	99%	100%	0%	0%
		Deposition = Erosion * Trap Efficiency (Filling)	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)		
		Deposition = Erosion * Trap Efficiency (Full)	(5,529,721)	(59,289)	(142,712)	(493,124)	(911,256)	(946,693)	(729,760)	(327,586)	(363,539)	(510,953)	(521,172)	(523,638)		
		Deposition (Filling)	51%	(1,481,136)	(2,106,437)	(2,414,426)	(1,611,751)	(940,188)	(537,250)	(201,469)	(201,469)	(268,625)	(268,625)	(268,625)		
		Deposition (Full)	49%	(28,892)	(69,545)	(240,304)	(444,065)	(461,334)	(355,620)	(159,636)	(177,157)	(248,993)	(253,973)	(255,174)		
Sources	Total Erosion	Sum of Debris Avalanche Erosion	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953	0	0
Sinks	Total Deposition Behind GBS	Sum of Deposition	(12,994,693)	(1,510,028)	(2,175,982)	(2,654,730)	(2,055,815)	(1,401,522)	(892,870)	(361,105)	(378,626)	(517,618)	(522,598)	(523,799)	0	0
Output from SRS	Output to North Fork Toutle River	Erosion - Deposition	13,202,963	6,611,245	2,801,573	2,060,848	1,087,903	432,314	155,036	31,860	14,339	6,335	1,355	154	0	0
North Fork Toutle River: SRS to Toutle River																
Input	Output from SRS		13,202,963	6,611,245	2,801,573	2,060,848	1,087,903	432,314	155,036	31,860	14,339	6,335	1,355	154	0	0
Sources	Bank Erosion North Fork Toutle	Est. & pro-rated from 99-06 Aerial Photos	94,617	3,270	4,495	10,951	20,083	17,117	12,395	7,115	4,930	6,274	3,634	4,353	0	
	Green River	Estimate from USGS Gage Data + 18% Unmeasured	158,366	69,401	21,569	28,007	24,571	11,298	3,519							
Sinks																
Output	Output to Toutle River		13,455,946	6,683,916	2,827,637	2,099,806	1,132,558	460,729	170,951	38,975	19,270	12,609	4,989	4,507	0	0
South Fork Toutle River: Upstream of USGS Gage																
Input	Upstream Source = Gage - Bank Erosion	Upstream Source Data Unavaliable	4,528,203	1,266,759	823,619	1,346,629	981,867	179,346	-9,485	-20,216	-12,623	-12,841	-9,802	-5,052	0	0
Sources	Bank Erosion South Fork	Est. & pro-rated from 99-06 Aerial Photos	212,148	7,169	13,961	25,744	29,009	38,934	36,797	20,216	12,623	12,841	9,802	5,052	0	
Sinks																
Output	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,740,351	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	0	0	0	0	0	0	0
South Fork Toutle River: Downstream of USGS Gage																
Input	@ USGS Gage # 14241500 South Fork	USGS Gage + 25% Unmeasured	4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Sources																
Sinks																
Output	Output to Toutle River		4,800,884	1,273,928	837,580	1,372,373	1,010,876	218,280	27,313	20,216	12,623	12,841	9,802	5,052	0	0
Toutle River: Confluence of North Fork and South Fork to USGS Gage at Tower Road																
Input	Output from North Fork and South Fork		18,256,831	7,957,844	3,665,217	3,472,179	2,143,434	679,010	198,263	59,190	31,892	25,450	14,791	9,560	0	0
Sources	Toutle Bank Erosion Above Tower	Est. & pro-rated from 99-06 Aerial Photos	20,822	1,814	2,560	3,173	3,236	2,843	2,059	1,396	1,210	760	1,091	679	0	
Sinks																
Output at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	18,277,652	7,959,658	3,667,777	3,475,352	2,146,671	681,853	200,323	60,586	33,103	26,210	15,882	10,238	0	0
Toutle River: USGS Gage at Tower Road to Cowlitz River																
Input at Tower Rd	@ USGS Gage # 14242580 Toutle at Tower Rd	Compare Sediment Budget to Gage Data	18,277,652	7,959,658	3,667,777	3,475,352	2,146,671	681,853	200,323	60,586	33,103	26,210	15,882	10,238	0	0
Sources	Toutle Bank Erosion Below Tower	LT1 Bank Stab - Reduce Bank Erosion 50%	46,017	1,961	2,881	6,460	13,606	7,677	3,640	2,145	1,850	2,353	3,207	238	0	
Sinks	SUMP at LT1 Up to 2.5 M Tons Capacity	Trap Efficiency		0%	0%	0%	0%	25%	72%	96%	100%	100%	100%	100%		
		Sink = Inflowing load* Trap Efficiency	(473,220)	0	0	0	0	(174,060)	(145,962)	(60,118)	(34,953)	(28,562)	(19,089)	(10,476)		
Output	Output to Cowlitz River		17,850,450	7,961,619	3,670,658	3,481,813	2,160,276	515,470	58,000	2,614	0	0	0	0	0	0
Cowlitz River: Toutle River to Columbia River																
Input	Input from Toutle River		17,850,450	7,961,619	3,670,658	3,481,813	2,160,276	515,470	58,000	2,614	0	0	0	0	0	0
	Input from Upper Cowlitz		0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sources			0	0	0	0	0	0	0	0	0	0	0	0	0	0
Sinks	Cowlitz River Deposition/Erosion	Hydro-Survey Comparisons	(1,526,837)	(37,670)	(126,425)	(523,045)	(310,403)	(210,151)	(50,049)	(10,209)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	
Output	Output to Columbia River		16,323,613	7,923,949	3,544,232	2,958,768	1,849,874	305,319	7,952	(7,595)	(32,113)	(30,860)	(51,582)	(123,787)	(20,543)	0

(Note: Negative values indicate deposition or sinks, Positive values indicate erosion or sources)

Table 3.7 Annual Sediment Budget Output, Existing Conditions (No Action)

Water Year	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
Debris Avalanche Erosion (Tons)												
1999	11,377,532	3,527,035	2,161,731	2,047,956	1,479,079	682,652	455,101	170,663	170,663	227,551	227,551	227,551
2000	946,244	293,336	179,786	170,324	113,549	66,237	37,850	14,194	14,194	18,925	18,925	18,925
2001	384,289	119,129	73,015	69,172	46,115	26,900	15,372	5,764	5,764	7,686	7,686	7,686
2002	10,523,145	3,262,175	1,999,398	1,894,166	1,262,777	736,620	420,926	157,847	157,847	210,463	210,463	210,463
2003	8,092,556	2,508,692	1,537,586	1,456,660	971,107	566,479	323,702	121,388	121,388	161,851	161,851	161,851
2004	2,428,360	752,792	461,388	437,105	291,403	169,985	97,134	36,425	36,425	48,567	48,567	48,567
2005	2,394,765	742,377	455,005	431,058	287,372	167,634	95,791	35,921	35,921	47,895	47,895	47,895
2006	9,323,296	2,890,222	1,771,426	1,678,193	1,118,795	652,631	372,932	139,849	139,849	186,466	186,466	186,466
2007	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953
SRS Deposition (Tons): Existing												
1999	(8,534,135)	(1,463,972)	(1,690,507)	(3,424,806)	(915,065)	(305,814)	(143,954)	(79,235)	(114,328)	(145,240)	(127,983)	(123,230)
2000	2,838,613	349,253	348,076	694,755	385,612	193,850	95,247	53,762	77,314	155,165	216,111	269,467
2001	162,102	678	5,160	31,219	40,018	18,951	8,883	4,991	7,572	12,501	14,873	17,256
2002	(4,578,825)	(196,328)	(372,075)	(1,279,977)	(1,004,389)	(432,953)	(200,262)	(111,903)	(170,890)	(252,189)	(267,121)	(290,738)
2003	(3,454,201)	(148,107)	(280,688)	(965,597)	(757,697)	(326,614)	(151,075)	(84,418)	(128,917)	(190,248)	(201,512)	(219,329)
2004	(449,084)	(35,492)	(68,080)	(200,222)	(91,678)	(29,522)	(12,736)	(6,950)	(11,157)	(7,443)	3,285	10,910
2005	(449,084)	(35,492)	(68,080)	(200,222)	(91,678)	(29,522)	(12,736)	(6,950)	(11,157)	(7,443)	3,285	10,910
2006	(4,114,631)	(95,789)	(258,149)	(1,063,381)	(971,541)	(426,844)	(197,422)	(110,430)	(169,018)	(252,690)	(271,464)	(297,904)
2007	(8,788,236)	(171,129)	(512,847)	(2,210,898)	(2,100,692)	(929,618)	(430,350)	(240,825)	(368,467)	(555,638)	(602,679)	(665,093)
Output from SRS (Tons): Existing												
1999	4,220,247	2,063,063	471,224	0	564,014	376,838	311,148	91,428	56,335	82,310	99,567	104,321
2000	3,784,857	642,589	527,862	865,079	499,162	260,087	133,097	67,955	91,507	174,090	235,036	288,392
2001	546,391	119,807	78,175	100,391	86,133	45,852	24,255	10,755	13,336	20,187	22,559	24,942
2002	6,136,022	3,065,847	1,627,323	614,189	258,388	303,667	220,664	45,945	0	0	0	0
2003	4,771,419	2,360,585	1,256,898	491,063	213,409	239,865	172,627	36,971	0	0	0	0
2004	1,979,276	717,300	393,308	236,883	199,725	140,463	84,399	29,476	25,268	41,124	51,853	59,478
2005	1,945,681	706,886	386,925	230,835	195,694	138,112	83,055	28,972	24,764	40,452	51,181	58,806
2006	5,500,492	2,794,433	1,513,277	614,812	147,255	225,787	175,510	29,419	0	0	0	0
2007	17,660,971	7,950,144	4,464,708	2,504,680	1,043,027	904,218	617,557	152,140	24,498	0	0	0
Sediment Load @ Mouth of Toutle (Tons): Existing												
1999	6,273,648	2,549,814	787,273	521,572	989,005	502,521	349,758	132,733	84,473	113,098	125,900	117,500
2000	4,536,208	793,189	624,636	1,026,821	649,182	318,317	157,988	100,311	113,475	198,128	255,374	298,786
2001	726,677	136,577	90,963	124,206	120,976	67,738	37,126	27,544	24,819	32,886	33,252	30,590
2002	7,648,966	3,417,284	1,855,360	991,164	569,843	399,355	252,011	80,906	23,746	26,022	21,962	11,313
2003	5,301,485	2,447,319	1,313,341	588,354	318,121	291,857	200,107	67,764	20,989	23,537	18,881	11,215
2004	2,462,951	801,337	449,262	332,110	295,535	182,501	104,489	56,260	43,470	61,101	68,675	68,210
2005	2,473,811	806,803	452,897	342,167	301,817	181,198	102,452	53,229	41,291	58,645	66,501	66,814
2006	6,116,559	2,907,967	1,586,412	738,070	269,046	277,927	199,700	61,384	21,738	23,817	20,190	10,309
2007	22,827,694	9,302,479	5,336,673	3,932,105	2,129,006	1,169,110	670,123	185,156	46,962	24,580	20,940	10,561

Table 3.8 Annual Sediment Budget Output with Raised SRS

Water Year	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
Debris Avalanche Erosion (Tons)												
1999	11,377,532	3,527,035	2,161,731	2,047,956	1,479,079	682,652	455,101	170,663	170,663	227,551	227,551	227,551
2000	946,244	293,336	179,786	170,324	113,549	66,237	37,850	14,194	14,194	18,925	18,925	18,925
2001	384,289	119,129	73,015	69,172	46,115	26,900	15,372	5,764	5,764	7,686	7,686	7,686
2002	10,523,145	3,262,175	1,999,398	1,894,166	1,262,777	736,620	420,926	157,847	157,847	210,463	210,463	210,463
2003	8,092,556	2,508,692	1,537,586	1,456,660	971,107	566,479	323,702	121,388	121,388	161,851	161,851	161,851
2004	2,428,360	752,792	461,388	437,105	291,403	169,985	97,134	36,425	36,425	48,567	48,567	48,567
2005	2,394,765	742,377	455,005	431,058	287,372	167,634	95,791	35,921	35,921	47,895	47,895	47,895
2006	9,323,296	2,890,222	1,771,426	1,678,193	1,118,795	652,631	372,932	139,849	139,849	186,466	186,466	186,466
2007	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953
SRS Deposition (Tons): Raised SRS												
1999	(8,725,065)	(1,254,661)	(1,784,349)	(2,045,245)	(1,479,079)	(682,652)	(455,101)	(170,663)	(170,663)	(227,551)	(227,551)	(227,551)
2000	(725,644)	(104,347)	(148,400)	(170,098)	(113,549)	(66,237)	(37,850)	(14,194)	(14,194)	(18,925)	(18,925)	(18,925)
2001	(294,699)	(42,378)	(60,268)	(69,080)	(46,115)	(26,900)	(15,372)	(5,764)	(5,764)	(7,686)	(7,686)	(7,686)
2002	(8,069,864)	(1,160,443)	(1,650,355)	(1,891,659)	(1,262,777)	(736,620)	(420,926)	(157,847)	(157,847)	(210,463)	(210,463)	(210,463)
2003	(6,205,922)	(892,409)	(1,269,163)	(1,454,732)	(971,107)	(566,479)	(323,702)	(121,388)	(121,388)	(161,851)	(161,851)	(161,851)
2004	(1,862,232)	(267,788)	(380,842)	(436,526)	(291,403)	(169,985)	(97,134)	(36,425)	(36,425)	(48,567)	(48,567)	(48,567)
2005	(1,836,469)	(264,084)	(375,573)	(430,487)	(287,372)	(167,634)	(95,791)	(35,921)	(35,921)	(47,895)	(47,895)	(47,895)
2006	(7,149,737)	(1,028,130)	(1,462,181)	(1,675,972)	(1,118,795)	(652,631)	(372,932)	(139,849)	(139,849)	(186,466)	(186,466)	(186,466)
2007	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)
Output from SRS (Tons): Raised SRS												
1999	2,652,467	2,272,374	377,382	2,711	0	0	0	0	0	0	0	0
2000	220,600	188,988	31,386	225	0	0	0	0	0	0	0	0
2001	89,590	76,752	12,746	92	0	0	0	0	0	0	0	0
2002	2,453,282	2,101,732	349,043	2,507	0	0	0	0	0	0	0	0
2003	1,886,634	1,616,283	268,422	1,928	0	0	0	0	0	0	0	0
2004	566,128	485,003	80,546	579	0	0	0	0	0	0	0	0
2005	558,296	478,294	79,432	571	0	0	0	0	0	0	0	0
2006	2,173,559	1,862,092	309,245	2,221	0	0	0	0	0	0	0	0
2007	6,107,512	5,232,318	868,951	6,242	0	0	0	0	0	0	0	0
Sediment Load @ Mouth of Toutle (Tons): Raised SRS												
1999	4,705,868	2,759,125	693,431	524,283	424,991	125,683	38,611	41,305	28,138	30,788	26,333	13,180
2000	971,952	339,588	128,160	161,968	150,021	58,230	24,891	32,356	21,968	24,039	20,338	10,394
2001	269,876	93,522	25,535	23,906	34,843	21,886	12,871	16,789	11,483	12,699	10,693	5,648
2002	3,966,225	2,453,168	577,080	379,482	311,455	95,688	31,347	34,961	23,746	26,022	21,962	11,313
2003	2,416,700	1,703,017	324,865	99,219	104,711	51,992	27,479	30,793	20,989	23,537	18,881	11,215
2004	1,049,804	569,041	136,500	95,806	95,810	42,038	20,090	26,784	18,202	19,977	16,823	8,732
2005	1,086,427	578,211	145,403	111,902	106,124	43,086	19,397	24,257	16,527	18,193	15,320	8,008
2006	2,789,625	1,975,627	382,380	125,480	121,791	52,140	24,190	31,965	21,738	23,817	20,190	10,309
2007	11,274,235	6,584,653	1,740,916	1,433,668	1,085,979	264,892	52,566	33,017	22,464	24,580	20,940	10,561

Table 3.9 Annual Sediment Budget Output with Grade Control Structures

Water Year	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
Debris Avalanche Erosion (Tons)												
1999	11,377,532	3,527,035	2,161,731	2,047,956	1,479,079	682,652	455,101	170,663	170,663	227,551	227,551	227,551
2000	946,244	293,336	179,786	170,324	113,549	66,237	37,850	14,194	14,194	18,925	18,925	18,925
2001	384,289	119,129	73,015	69,172	46,115	26,900	15,372	5,764	5,764	7,686	7,686	7,686
2002	10,523,145	3,262,175	1,999,398	1,894,166	1,262,777	736,620	420,926	157,847	157,847	210,463	210,463	210,463
2003	8,092,556	2,508,692	1,537,586	1,456,660	971,107	566,479	323,702	121,388	121,388	161,851	161,851	161,851
2004	2,428,360	752,792	461,388	437,105	291,403	169,985	97,134	36,425	36,425	48,567	48,567	48,567
2005	2,394,765	742,377	455,005	431,058	287,372	167,634	95,791	35,921	35,921	47,895	47,895	47,895
2006	9,323,296	2,890,222	1,771,426	1,678,193	1,118,795	652,631	372,932	139,849	139,849	186,466	186,466	186,466
2007	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953
SRS Deposition (Tons): Grade Control Structures												
1999	(8,725,065)	(1,254,661)	(1,784,349)	(2,045,245)	(1,479,079)	(682,652)	(455,101)	(170,663)	(170,663)	(227,551)	(227,551)	(227,551)
2000	(725,644)	(104,347)	(148,400)	(170,098)	(113,549)	(66,237)	(37,850)	(14,194)	(14,194)	(18,925)	(18,925)	(18,925)
2001	(294,699)	(42,378)	(60,268)	(69,080)	(46,115)	(26,900)	(15,372)	(5,764)	(5,764)	(7,686)	(7,686)	(7,686)
2002	(8,069,864)	(1,160,443)	(1,650,355)	(1,891,659)	(1,262,777)	(736,620)	(420,926)	(157,847)	(157,847)	(210,463)	(210,463)	(210,463)
2003	(6,205,922)	(892,409)	(1,269,163)	(1,454,732)	(971,107)	(566,479)	(323,702)	(121,388)	(121,388)	(161,851)	(161,851)	(161,851)
2004	(1,862,232)	(267,788)	(380,842)	(436,526)	(291,403)	(169,985)	(97,134)	(36,425)	(36,425)	(48,567)	(48,567)	(48,567)
2005	(1,836,469)	(264,084)	(375,573)	(430,487)	(287,372)	(167,634)	(95,791)	(35,921)	(35,921)	(47,895)	(47,895)	(47,895)
2006	(7,149,737)	(1,028,130)	(1,462,181)	(1,675,972)	(1,118,795)	(652,631)	(372,932)	(139,849)	(139,849)	(186,466)	(186,466)	(186,466)
2007	(12,994,693)	(1,510,028)	(2,175,982)	(2,654,730)	(2,055,815)	(1,401,522)	(892,870)	(361,105)	(378,626)	(517,618)	(522,598)	(523,799)
Output from SRS (Tons): Grade Control Structures												
1999	2,652,467	2,272,374	377,382	2,711	0	0	0	0	0	0	0	0
2000	220,600	188,988	31,386	225	0	0	0	0	0	0	0	0
2001	89,590	76,752	12,746	92	0	0	0	0	0	0	0	0
2002	2,453,282	2,101,732	349,043	2,507	0	0	0	0	0	0	0	0
2003	1,886,634	1,616,283	268,422	1,928	0	0	0	0	0	0	0	0
2004	566,128	485,003	80,546	579	0	0	0	0	0	0	0	0
2005	558,296	478,294	79,432	571	0	0	0	0	0	0	0	0
2006	2,173,559	1,862,092	309,245	2,221	0	0	0	0	0	0	0	0
2007	13,202,963	6,611,245	2,801,573	2,060,848	1,087,903	432,314	155,036	31,860	14,339	6,335	1,355	154
Sediment Load @ Mouth of Toutle (Tons): Grade Control Structures												
1999	4,705,868	2,759,125	693,431	524,283	424,991	125,683	38,611	41,305	28,138	30,788	26,333	13,180
2000	971,952	339,588	128,160	161,968	150,021	58,230	24,891	32,356	21,968	24,039	20,338	10,394
2001	269,876	93,522	25,535	23,906	34,843	21,886	12,871	16,789	11,483	12,699	10,693	5,648
2002	3,966,225	2,453,168	577,080	379,482	311,455	95,688	31,347	34,961	23,746	26,022	21,962	11,313
2003	2,416,700	1,703,017	324,865	99,219	104,711	51,992	27,479	30,793	20,989	23,537	18,881	11,215
2004	1,049,804	569,041	136,500	95,806	95,810	42,038	20,090	26,784	18,202	19,977	16,823	8,732
2005	1,086,427	578,211	145,403	111,902	106,124	43,086	19,397	24,257	16,527	18,193	15,320	8,008
2006	2,789,625	1,975,627	382,380	125,480	121,791	52,140	24,190	31,965	21,738	23,817	20,190	10,309
2007	18,369,687	7,963,580	3,673,538	3,488,273	2,173,882	697,207	207,602	64,876	36,803	30,915	22,295	10,714

Table 3.10 Annual Sediment Budget Output with LT1 Sump

Water Year	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
Debris Avalanche Erosion (Tons)												
1999	11,377,532	3,527,035	2,161,731	2,047,956	1,479,079	682,652	455,101	170,663	170,663	227,551	227,551	227,551
2000	946,244	293,336	179,786	170,324	113,549	66,237	37,850	14,194	14,194	18,925	18,925	18,925
2001	384,289	119,129	73,015	69,172	46,115	26,900	15,372	5,764	5,764	7,686	7,686	7,686
2002	10,523,145	3,262,175	1,999,398	1,894,166	1,262,777	736,620	420,926	157,847	157,847	210,463	210,463	210,463
2003	8,092,556	2,508,692	1,537,586	1,456,660	971,107	566,479	323,702	121,388	121,388	161,851	161,851	161,851
2004	2,428,360	752,792	461,388	437,105	291,403	169,985	97,134	36,425	36,425	48,567	48,567	48,567
2005	2,394,765	742,377	455,005	431,058	287,372	167,634	95,791	35,921	35,921	47,895	47,895	47,895
2006	9,323,296	2,890,222	1,771,426	1,678,193	1,118,795	652,631	372,932	139,849	139,849	186,466	186,466	186,466
2007	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953
SRS Deposition (Tons): LT1 Sump												
1999	(8,534,135)	(1,463,972)	(1,690,507)	(3,424,806)	(915,065)	(305,814)	(143,954)	(79,235)	(114,328)	(145,240)	(127,983)	(123,230)
2000	(1,180,031)	(14,998)	(105,731)	(444,659)	(308,189)	(113,979)	(50,246)	(27,743)	(44,192)	(44,263)	(20,721)	(5,310)
2001	0	0	0	0	0	0	0	0	0	0	0	0
2002	(4,578,825)	(196,328)	(372,075)	(1,279,977)	(1,004,389)	(432,953)	(200,262)	(111,903)	(170,890)	(252,189)	(267,121)	(290,738)
2003	(3,454,201)	(148,107)	(280,688)	(965,597)	(757,697)	(326,614)	(151,075)	(84,418)	(128,917)	(190,248)	(201,512)	(219,329)
2004	(570,483)	(35,999)	(71,944)	(223,602)	(121,648)	(43,715)	(19,388)	(10,688)	(16,828)	(16,805)	(7,853)	(2,012)
2005	(570,483)	(35,999)	(71,944)	(223,602)	(121,648)	(43,715)	(19,388)	(10,688)	(16,828)	(16,805)	(7,853)	(2,012)
2006	(4,114,631)	(95,789)	(258,149)	(1,063,381)	(971,541)	(426,844)	(197,422)	(110,430)	(169,018)	(252,690)	(271,464)	(297,904)
2007	(8,788,236)	(171,129)	(512,847)	(2,210,898)	(2,100,692)	(929,618)	(430,350)	(240,825)	(368,467)	(555,638)	(602,679)	(665,093)
Output from SRS (Tons): LT1 Sump												
1999	4,220,247	2,063,063	471,224	0	564,014	376,838	311,148	91,428	56,335	82,310	99,567	104,321
2000	3,784,857	642,589	527,862	865,079	499,162	260,087	133,097	67,955	91,507	174,090	235,036	288,392
2001	546,391	119,807	78,175	100,391	86,133	45,852	24,255	10,755	13,336	20,187	22,559	24,942
2002	6,136,022	3,065,847	1,627,323	614,189	258,388	303,667	220,664	45,945	0	0	0	0
2003	4,771,419	2,360,585	1,256,898	491,063	213,409	239,865	172,627	36,971	0	0	0	0
2004	1,979,276	717,300	393,308	236,883	199,725	140,463	84,399	29,476	25,268	41,124	51,853	59,478
2005	1,945,681	706,886	386,925	230,835	195,694	138,112	83,055	28,972	24,764	40,452	51,181	58,806
2006	5,500,492	2,794,433	1,513,277	614,812	147,255	225,787	175,510	29,419	0	0	0	0
2007	17,660,971	7,950,144	4,464,708	2,504,680	1,043,027	904,218	617,557	152,140	24,498	0	0	0
Sediment Load @ Mouth of Toutle (Tons): LT1 Sump												
1999	5,287,649	2,547,303	783,585	513,302	971,587	368,321	98,135	5,416	0	0	0	0
2000	3,351,002	791,344	621,926	1,020,742	636,379	232,563	43,953	4,096	0	0	0	0
2001	518,738	135,570	89,483	120,886	113,984	47,689	10,026	1,102	0	0	0	0
2002	7,175,057	3,415,298	1,852,443	984,622	556,066	292,733	70,616	3,281	0	0	0	0
2003	4,920,386	2,445,797	1,311,105	583,340	307,561	213,728	56,100	2,754	0	0	0	0
2004	2,022,135	799,819	447,032	327,108	285,001	131,988	28,912	2,275	0	0	0	0
2005	2,047,698	805,395	450,830	337,532	292,056	131,340	28,391	2,154	0	0	0	0
2006	5,738,442	2,906,105	1,583,676	731,936	256,127	202,320	55,805	2,473	0	0	0	0
2007	21,740,756	9,300,518	5,333,792	3,925,645	2,115,400	868,249	189,526	7,625	0	0	0	0

Table 3.11 Annual Sediment Budget Output with Raised SRS and LT1 Sump

Water Year	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
Debris Avalanche Erosion (Tons)												
1999	11,377,532	3,527,035	2,161,731	2,047,956	1,479,079	682,652	455,101	170,663	170,663	227,551	227,551	227,551
2000	946,244	293,336	179,786	170,324	113,549	66,237	37,850	14,194	14,194	18,925	18,925	18,925
2001	384,289	119,129	73,015	69,172	46,115	26,900	15,372	5,764	5,764	7,686	7,686	7,686
2002	10,523,145	3,262,175	1,999,398	1,894,166	1,262,777	736,620	420,926	157,847	157,847	210,463	210,463	210,463
2003	8,092,556	2,508,692	1,537,586	1,456,660	971,107	566,479	323,702	121,388	121,388	161,851	161,851	161,851
2004	2,428,360	752,792	461,388	437,105	291,403	169,985	97,134	36,425	36,425	48,567	48,567	48,567
2005	2,394,765	742,377	455,005	431,058	287,372	167,634	95,791	35,921	35,921	47,895	47,895	47,895
2006	9,323,296	2,890,222	1,771,426	1,678,193	1,118,795	652,631	372,932	139,849	139,849	186,466	186,466	186,466
2007	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953
SRS Deposition (Tons): Raised SRS + LT1 Sump												
1999	(8,725,065)	(1,254,661)	(1,784,349)	(2,045,245)	(1,479,079)	(682,652)	(455,101)	(170,663)	(170,663)	(227,551)	(227,551)	(227,551)
2000	(725,644)	(104,347)	(148,400)	(170,098)	(113,549)	(66,237)	(37,850)	(14,194)	(14,194)	(18,925)	(18,925)	(18,925)
2001	(294,699)	(42,378)	(60,268)	(69,080)	(46,115)	(26,900)	(15,372)	(5,764)	(5,764)	(7,686)	(7,686)	(7,686)
2002	(8,069,864)	(1,160,443)	(1,650,355)	(1,891,659)	(1,262,777)	(736,620)	(420,926)	(157,847)	(157,847)	(210,463)	(210,463)	(210,463)
2003	(6,205,922)	(892,409)	(1,269,163)	(1,454,732)	(971,107)	(566,479)	(323,702)	(121,388)	(121,388)	(161,851)	(161,851)	(161,851)
2004	(1,862,232)	(267,788)	(380,842)	(436,526)	(291,403)	(169,985)	(97,134)	(36,425)	(36,425)	(48,567)	(48,567)	(48,567)
2005	(1,836,469)	(264,084)	(375,573)	(430,487)	(287,372)	(167,634)	(95,791)	(35,921)	(35,921)	(47,895)	(47,895)	(47,895)
2006	(7,149,737)	(1,028,130)	(1,462,181)	(1,675,972)	(1,118,795)	(652,631)	(372,932)	(139,849)	(139,849)	(186,466)	(186,466)	(186,466)
2007	(20,090,144)	(2,888,955)	(4,108,603)	(4,709,336)	(3,143,719)	(1,833,836)	(1,047,906)	(392,965)	(392,965)	(523,953)	(523,953)	(523,953)
Output from SRS (Tons): Raised SRS + LT1 Sump												
1999	2,652,467	2,272,374	377,382	2,711	0	0	0	0	0	0	0	0
2000	220,600	188,988	31,386	225	0	0	0	0	0	0	0	0
2001	89,590	76,752	12,746	92	0	0	0	0	0	0	0	0
2002	2,453,282	2,101,732	349,043	2,507	0	0	0	0	0	0	0	0
2003	1,886,634	1,616,283	268,422	1,928	0	0	0	0	0	0	0	0
2004	566,128	485,003	80,546	579	0	0	0	0	0	0	0	0
2005	558,296	478,294	79,432	571	0	0	0	0	0	0	0	0
2006	2,173,559	1,862,092	309,245	2,221	0	0	0	0	0	0	0	0
2007	6,107,512	5,232,318	868,951	6,242	0	0	0	0	0	0	0	0
Sediment Load @ Mouth of Toutle (Tons): Raised SRS + LT1 Sump												
1999	4,467,814	2,756,614	689,744	516,013	407,573	86,609	9,655	1,607	0	0	0	0
2000	801,797	337,743	125,449	155,889	137,217	38,131	6,104	1,264	0	0	0	0
2001	182,200	92,514	24,055	20,586	27,851	13,412	3,128	654	0	0	0	0
2002	3,770,917	2,451,182	574,163	372,940	297,678	65,722	7,866	1,366	0	0	0	0
2003	2,255,119	1,701,495	322,630	94,205	94,151	34,413	7,011	1,214	0	0	0	0
2004	910,813	567,522	134,270	90,804	85,275	26,982	4,912	1,047	0	0	0	0
2005	957,582	576,804	143,337	107,267	96,362	28,092	4,773	947	0	0	0	0
2006	2,622,298	1,973,765	379,644	119,345	108,872	33,529	5,896	1,247	0	0	0	0
2007	11,027,793	6,582,692	1,738,036	1,427,207	1,072,373	192,286	13,913	1,286	0	0	0	0

Table 3.12 Annual Sediment Budget Output with Grade Control Structures and LT1

Water Year	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63
Debris Avalanche Erosion (Tons)												
1999	11,377,532	3,527,035	2,161,731	2,047,956	1,479,079	682,652	455,101	170,663	170,663	227,551	227,551	227,551
2000	946,244	293,336	179,786	170,324	113,549	66,237	37,850	14,194	14,194	18,925	18,925	18,925
2001	384,289	119,129	73,015	69,172	46,115	26,900	15,372	5,764	5,764	7,686	7,686	7,686
2002	10,523,145	3,262,175	1,999,398	1,894,166	1,262,777	736,620	420,926	157,847	157,847	210,463	210,463	210,463
2003	8,092,556	2,508,692	1,537,586	1,456,660	971,107	566,479	323,702	121,388	121,388	161,851	161,851	161,851
2004	2,428,360	752,792	461,388	437,105	291,403	169,985	97,134	36,425	36,425	48,567	48,567	48,567
2005	2,394,765	742,377	455,005	431,058	287,372	167,634	95,791	35,921	35,921	47,895	47,895	47,895
2006	9,323,296	2,890,222	1,771,426	1,678,193	1,118,795	652,631	372,932	139,849	139,849	186,466	186,466	186,466
2007	26,197,656	8,121,273	4,977,555	4,715,578	3,143,719	1,833,836	1,047,906	392,965	392,965	523,953	523,953	523,953
SRS Deposition (Tons): Grade Control Structures + LT1 Sump												
1999	(8,725,065)	(1,254,661)	(1,784,349)	(2,045,245)	(1,479,079)	(682,652)	(455,101)	(170,663)	(170,663)	(227,551)	(227,551)	(227,551)
2000	(725,644)	(104,347)	(148,400)	(170,098)	(113,549)	(66,237)	(37,850)	(14,194)	(14,194)	(18,925)	(18,925)	(18,925)
2001	(294,699)	(42,378)	(60,268)	(69,080)	(46,115)	(26,900)	(15,372)	(5,764)	(5,764)	(7,686)	(7,686)	(7,686)
2002	(8,069,864)	(1,160,443)	(1,650,355)	(1,891,659)	(1,262,777)	(736,620)	(420,926)	(157,847)	(157,847)	(210,463)	(210,463)	(210,463)
2003	(6,205,922)	(892,409)	(1,269,163)	(1,454,732)	(971,107)	(566,479)	(323,702)	(121,388)	(121,388)	(161,851)	(161,851)	(161,851)
2004	(1,862,232)	(267,788)	(380,842)	(436,526)	(291,403)	(169,985)	(97,134)	(36,425)	(36,425)	(48,567)	(48,567)	(48,567)
2005	(1,836,469)	(264,084)	(375,573)	(430,487)	(287,372)	(167,634)	(95,791)	(35,921)	(35,921)	(47,895)	(47,895)	(47,895)
2006	(7,149,737)	(1,028,130)	(1,462,181)	(1,675,972)	(1,118,795)	(652,631)	(372,932)	(139,849)	(139,849)	(186,466)	(186,466)	(186,466)
2007	(12,994,693)	(1,510,028)	(2,175,982)	(2,654,730)	(2,055,815)	(1,401,522)	(892,870)	(361,105)	(378,626)	(517,618)	(522,598)	(523,799)
Output from SRS (Tons): Grade Control Structures + LT1 Sump												
1999	2,652,467	2,272,374	377,382	2,711	0	0	0	0	0	0	0	0
2000	220,600	188,988	31,386	225	0	0	0	0	0	0	0	0
2001	89,590	76,752	12,746	92	0	0	0	0	0	0	0	0
2002	2,453,282	2,101,732	349,043	2,507	0	0	0	0	0	0	0	0
2003	1,886,634	1,616,283	268,422	1,928	0	0	0	0	0	0	0	0
2004	566,128	485,003	80,546	579	0	0	0	0	0	0	0	0
2005	558,296	478,294	79,432	571	0	0	0	0	0	0	0	0
2006	2,173,559	1,862,092	309,245	2,221	0	0	0	0	0	0	0	0
2007	13,202,963	6,611,245	2,801,573	2,060,848	1,087,903	432,314	155,036	31,860	14,339	6,335	1,355	154
Sediment Load @ Mouth of Toutle (Tons): Grade Control Structures + LT1												
1999	4,467,814	2,756,614	689,744	516,013	407,573	86,609	9,655	1,607	0	0	0	0
2000	801,797	337,743	125,449	155,889	137,217	38,131	6,104	1,264	0	0	0	0
2001	182,200	92,514	24,055	20,586	27,851	13,412	3,128	654	0	0	0	0
2002	3,770,917	2,451,182	574,163	372,940	297,678	65,722	7,866	1,366	0	0	0	0
2003	2,255,119	1,701,495	322,630	94,205	94,151	34,413	7,011	1,214	0	0	0	0
2004	910,813	567,522	134,270	90,804	85,275	26,982	4,912	1,047	0	0	0	0
2005	957,582	576,804	143,337	107,267	96,362	28,092	4,773	947	0	0	0	0
2006	2,622,298	1,973,765	379,644	119,345	108,872	33,529	5,896	1,247	0	0	0	0
2007	17,850,450	7,961,619	3,670,658	3,481,813	2,160,276	515,470	58,000	2,614	0	0	0	0

A range of projections of the four scenarios of measures (raised SRS, raised SRS + LT1, grade control structures, and grade control structures + LT1) was conducted by combining the annual sediment budgets in combination to represent forecasting years 2008 – 2035. The five forecasting sequences of annual water years presented in Table 1.1 were used (maximum, 5%, 50%, 95%, and minimum sequences). Each forecasting sequence incorporates that dates of operation of each measure as well as the capacity of each measure. A more specific description of each scenario forecasting is provided below.

Raised SRS

Forecasting the performance of the raised SRS during the years 2008 – 2014 is a replication of the existing condition sediment budgets. The raised SRS measure becomes operational in 2015, therefore, forecasting years 2015 through 2035 use the raised SRS annual sediment budgets. The cumulative deposition calculated behind the SRS by 2035 is check to determine if capacity is exceeded. Deposition occurring behind the SRS by 2035 ranges from 78 – 275 M Tons; well under the capacity of 641 M Tons.

Raised SRS + LT1

Operation of the LT1 sump and raised SRS begin in water years 2011 and 2015, respectively. Therefore, forecasting years 2008 – 2010 reference existing condition sediment budgets, years 2011 – 2014 reference the LT1 sediment budgets, and years 2015 – 2035 reference the raised SRS + LT1 budgets. The cumulative deposition in the LT1 sump by 2035 ranges from 4.2 to 5.2 M Tons.

Grade Control Structures

The first set of grade control structures become operational at the beginning of water year 2012. Therefore, forecasting years 2008 – 2011 reference the existing sediment budgets. The grade control structures sediment budgets are referenced starting in forecast year 2012. The cumulative deposition behind the structures is calculated through the forecast period to determine when additional sets of structures need to be built. An additional set of 10 structures is constructed if the cumulative deposition is between 7.7 and 10.3 M Tons. Table 3.13 provides the cumulative deposition behind the grade control structures by 2035 and a range of the number of sets that are required for each forecasting sequence.

Table 3.13 Grade Control Structures Installation Sequence

Forecast Sequence	Cumulative Deposition by 2035[A] (M Tons)	# of Structure Sets [B]	
		7.7 M Tons	10.3 M Tons
Maximum	181	23	17
5% Exceedance	143	18	13
50% Exceedance	121	15	11
95% Exceedance	114	14	11
Minimum	77	10	7

[A] Cumulative deposition behind grade control structures starting in 2012.
[B] Number of grade control structure sets of 10 needed assuming structures fill to a maximum of 7.7 M Tons or 10.3 M Tons (6 MCY or 8 MCY).

Grade Control Structures + LT1

Operation of the LT1 sump and grade control structures begin in water years 2011 and 2012, respectively. Therefore, forecasting years 2008 – 2010 reference existing condition sediment budgets, 2011 reference the LT1 sediment budgets, and years 2015 – 2035 reference the grade control structures + LT1 budgets. The cumulative deposition in the LT1 sump by 2035 ranges from 2.8 to 11.2 M Tons.

4.0 Forecasting Results

Results of the forecasting of the cumulative sediment load at the mouth of the Toutle River are shown graphically in Figure 4.1.

Table 4.1 summarizes the range of the cumulative sediment output from the SRS in 2035 for each measure analyzed. Similarly, Table 4.2 and Figure 4.2 summarize the range of the cumulative sediment load at the mouth of the Toutle River in 2035.

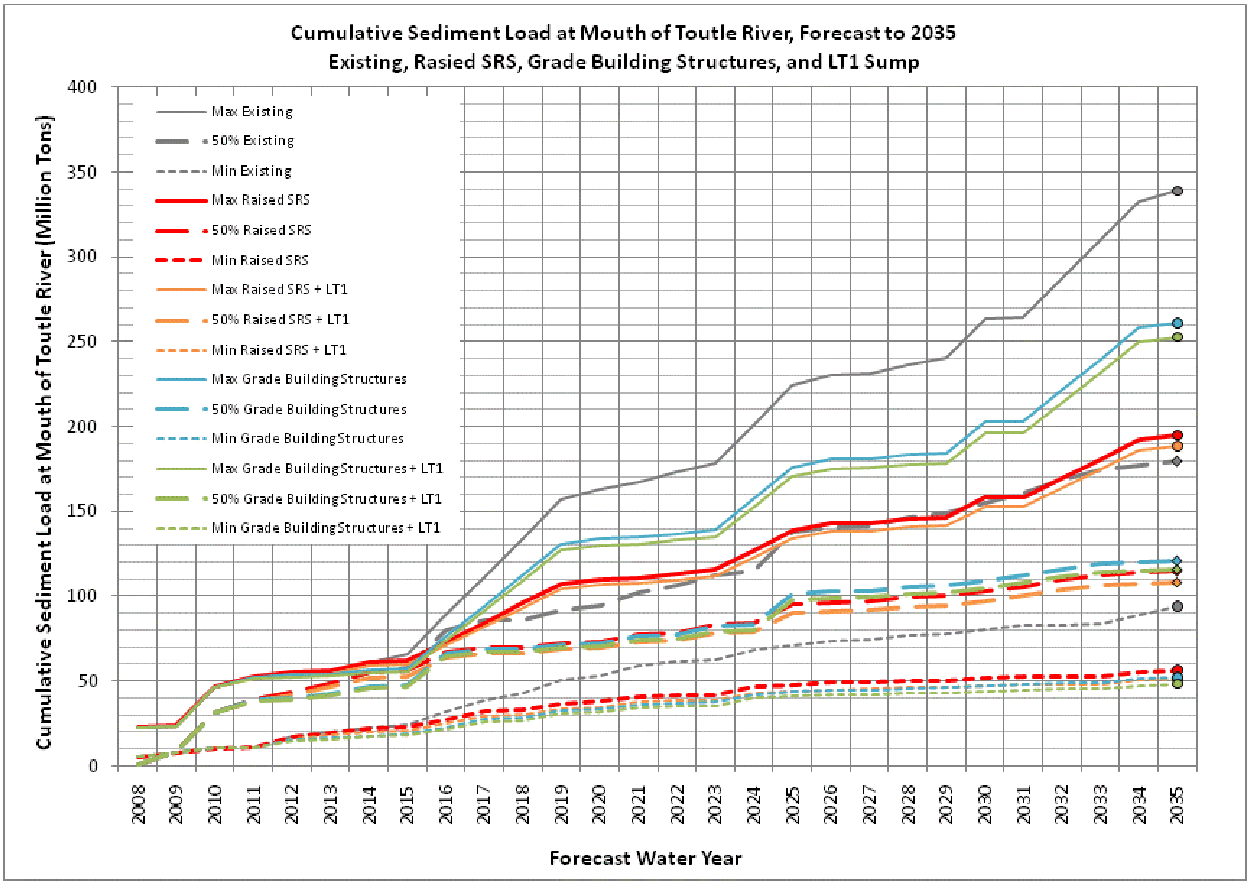


Figure 4.1 Forecast of Cumulative Sediment Load at Mouth of Toutle River by Forecasting Year for Existing, Raised SRS, Grade Control Structures, and LT1.

Table 4.1 Comparison of Cumulative Output from SRS in 2035 (all grain classes)

Measure	Cumulative Output from SRS in 2035 (Million Tons)				
	Max	5%	50%	95%	Min
Existing	266.0	187.1	144.5	106.7	74.4
Raised SRS	122.0	102.2	79.5	54.9	37.1
Raised SRS + LT1	--	--	--	--	--
Grade Control Structures	188.2	121.7	85.7	48.4	32.7
Grade Control Structures + LT1	--	--	--	--	--

Table 4.2 Comparison of Cumulative Sediment Load at Mouth of Toutle in 2035 (all grain classes)

Measure	Cumulative Sediment Load at Mouth of Toutle River in 2035 (Million Tons)				
	Max	**5%**	**50%**	**95%**	**Min**
Existing	338.9	238.9	179.7	131.8	93.9
Raised SRS	194.9	154.0	114.7	80.0	56.5
Raised SRS + LT1	188.7	147.2	108.3	73.0	51.4
Grade Control Structures	261.1	173.4	120.9	73.4	52.1
Grade Control Structures + LT1	252.8	166.9	115.9	68.5	48.4

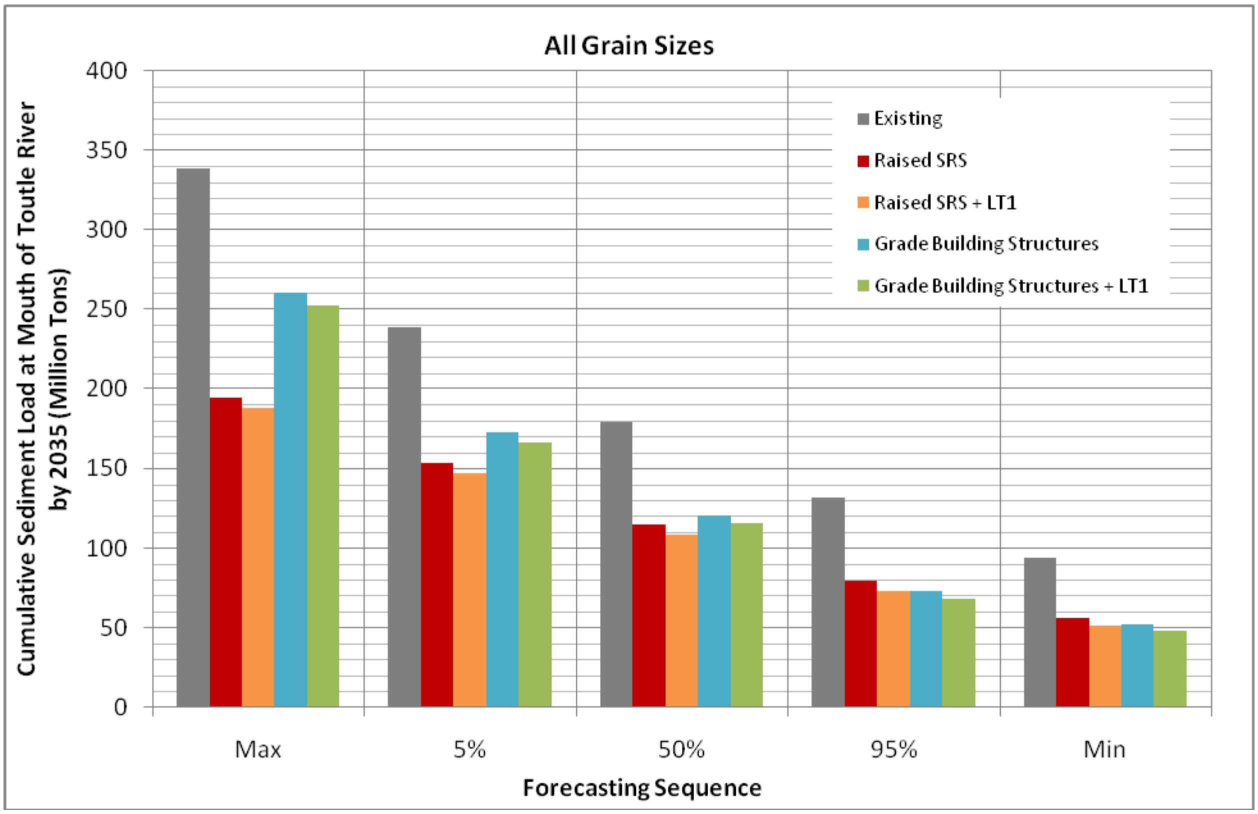


Figure 4.2 Cumulative Sediment Load at Mouth of Toutle River by 2035 for selected forecasting sequences and all Grain Classes

The raised SRS measure decreases the cumulative sediment load at the mouth of the Toutle River between 36% and 40%. The raised SRS measure paired with the LT1 sump decreases the load from 38% to 44%. The addition of the LT1 sump and bank stabilization increases the sediment load reduction by only 2% to 4%.

The grade control structures decreases the cumulative sediment load at the mouth of the Toutle River by 23% to 44%. With the addition of the LT1 sump the decrease in sediment load ranges from 25% to 48%, only 2% to 4%.

Results of the forecasting were further analyzed by grain size. Material in the range of 0.125 to 2 mm is linked to depositional problems in the lower Cowlitz River. Forecast of results of the cumulative output from the SRS and sediment load at the mouth of the Toutle River by 2035 per grain class are compared for each measure in Tables 4.3 though 4.7.

Table 4.3 Comparison of Measures to Existing (No Action) Cumulative Output from the SRS and Sediment Load at the Mouth of the Toutle River by 2035 for the Maximum Forecasting Sequence

Measure	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
	Cumulative Output from SRS by 2035 (Million Tons): Maximum Sequence											
Existing	266.0	117.7	64.9	36.5	16.8	14.1	9.7	2.5	0.8	0.8	1.0	1.2
Raised SRS	122.0	85.2	21.8	6.0	3.2	2.7	1.9	0.5	0.2	0.2	0.2	0.2
Raised SRS + LT1	--	--	--	--	--	--	--	--	--	--	--	--
Grade Control Structures	188.2	97.9	39.8	25.7	13.6	6.6	3.1	0.7	0.3	0.2	0.1	0.1
Grade Control Structures + LT1	--	--	--	--	--	--	--	--	--	--	--	--
	% Difference Relative to Existing											
Raised SRS	-54%	-28%	-66%	-84%	-81%	-81%	-81%	-81%	-79%	-80%	-81%	-83%
Grade Control Structures	-29%	-17%	-39%	-29%	-19%	-54%	-68%	-71%	-66%	-79%	-87%	-89%

Measure	Total	CM 0.0625	VFS 0.125	FS 0.25	MS 0.5	CS 1	VCS 2	VFG 4	FG 8	MG 16	CG 32	VCG 63
	Cumulative Sediment Load at Mouth of Toutle River by 2035 (Million Tons): Maximum Sequence											
Existing	338.9	136.1	76.7	56.0	32.1	18.2	10.7	3.3	1.3	1.4	1.6	1.5
Raised SRS	194.9	103.5	33.7	25.5	18.5	6.7	2.9	1.3	0.7	0.8	0.7	0.5
Raised SRS + LT1	188.66	103.5	33.6	25.4	18.2	5.5	1.8	0.4	0.1	0.1	0.1	0.1
Grade Control Structures	261.1	116.3	51.6	45.2	28.8	10.6	4.1	1.6	0.8	0.8	0.7	0.4
Grade Control Structures + LT1	252.77	116.3	51.6	45.1	28.5	8.4	2.1	0.4	0.1	0.1	0.1	0.1
	% Difference Relative to Existing											
Raised SRS	-42%	-24%	-56%	-54%	-42%	-63%	-73%	-60%	-45%	-44%	-54%	-67%
Raised SRS + LT1	-44%	-24%	-56%	-55%	-43%	-70%	-83%	-87%	-91%	-94%	-95%	-97%
Grade Control Structures	-23%	-15%	-33%	-19%	-10%	-42%	-61%	-53%	-37%	-44%	-57%	-73%
Grade Control Structures + LT1	-25%	-15%	-33%	-19%	-11%	-54%	-80%	-87%	-91%	-94%	-95%	-97%

Table 4.4 Comparison of Measures to Existing (No Action) Cumulative Output from the SRS and Sediment Load at the Mouth of the Toutle River by 2035 for the 5% Exceedance Forecasting Sequence

Measure	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Output from SRS by 2035 (Million Tons): 5% Exceedance											
Existing	187.1	81.7	43.3	23.9	13.0	10.5	7.2	2.0	0.9	1.2	1.6	1.9
Raised SRS	102.2	65.0	20.6	7.0	3.4	3.1	2.2	0.5	0.1	0.1	0.1	0.1
Raised SRS + LT1	--	--	--	--	--	--	--	--	--	--	--	--
Grade Control Structures	121.7	68.8	25.6	14.0	6.7	3.8	2.1	0.5	0.1	0.0	0.0	0.0
Grade Control Structures + LT1	--	--	--	--	--	--	--	--	--	--	--	--
	% Difference Relative to Existing											
Raised SRS	-45%	-20%	-52%	-71%	-74%	-71%	-70%	-73%	-86%	-92%	-92%	-93%
Grade Control Structures	-35%	-16%	-41%	-41%	-49%	-64%	-72%	-76%	-86%	-96%	-98%	-99%

Measure	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Sediment Load at Mouth of Toutle River by 2035 (Million Tons): 5% Exceedance											
Existing	238.9	94.2	51.4	37.3	23.7	13.6	8.1	2.8	1.5	1.9	2.2	2.2
Raised SRS	154.0	77.6	28.7	20.4	14.1	6.1	3.1	1.4	0.7	0.8	0.7	0.4
Raised SRS + LT1	147.2	77.6	28.6	20.2	13.8	4.8	1.5	0.3	0.1	0.1	0.1	0.1
Grade Control Structures	173.4	81.4	33.8	27.4	17.4	6.9	2.9	1.3	0.7	0.7	0.6	0.3
Grade Control Structures + LT1	166.9	81.4	33.7	27.2	17.1	5.4	1.5	0.3	0.1	0.1	0.1	0.1
	% Difference Relative to Existing											
Raised SRS	-36%	-18%	-44%	-45%	-40%	-55%	-62%	-51%	-52%	-60%	-69%	-81%
Raised SRS + LT1	-38%	-18%	-44%	-46%	-42%	-64%	-82%	-89%	-94%	-96%	-97%	-98%
Grade Control Structures	-27%	-14%	-34%	-27%	-27%	-49%	-64%	-53%	-51%	-63%	-73%	-86%
Grade Control Structures + LT1	-30%	-14%	-34%	-27%	-28%	-60%	-82%	-89%	-94%	-96%	-97%	-98%

Table 4.5 Comparison of Measures to Existing (No Action) Cumulative Output from the SRS and Sediment Load at the Mouth of the Toutle River by 2035 for the 50% Exceedance Forecasting Sequence

Measure	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Output from SRS by 2035 (Million Tons): 50% Exceedance											
Existing	144.5	64.7	34.4	17.5	9.2	8.0	5.5	1.4	0.6	0.9	1.1	1.3
Raised SRS	79.5	50.4	15.2	5.2	2.9	2.4	1.7	0.5	0.2	0.3	0.4	0.4
Raised SRS + LT1	--	--	--	--	--	--	--	--	--	--	--	--
Grade Control Structures	85.7	52.1	17.5	8.0	3.8	2.4	1.4	0.3	0.1	0.0	0.0	0.0
Grade Control Structures + LT1	--	--	--	--	--	--	--	--	--	--	--	--
	% Difference Relative to Existing											
Raised SRS	-45%	-22%	-56%	-70%	-68%	-69%	-69%	-69%	-68%	-68%	-67%	-67%
Grade Control Structures	-41%	-19%	-49%	-54%	-59%	-70%	-75%	-78%	-89%	-96%	-98%	-98%

Measure	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Sediment Load at Mouth of Toutle River by 2035 (Million Tons): 50% Exceedance											
Existing	179.7	72.8	39.7	26.2	16.5	10.2	6.3	2.3	1.2	1.5	1.6	1.6
Raised SRS	114.7	58.5	20.5	13.9	10.2	4.7	2.5	1.3	0.8	0.9	0.9	0.7
Raised SRS + LT1	108.3	58.4	20.4	13.8	9.9	3.8	1.4	0.3	0.1	0.1	0.1	0.1
Grade Control Structures	120.9	60.2	22.8	16.7	11.1	4.7	2.2	1.2	0.6	0.7	0.6	0.3
Grade Control Structures + LT1	115.9	60.2	22.7	16.5	10.8	3.8	1.3	0.3	0.1	0.1	0.1	0.1
	% Difference Relative to Existing											
Raised SRS	-36%	-20%	-48%	-47%	-38%	-54%	-61%	-43%	-35%	-39%	-46%	-55%
Raised SRS + LT1	-40%	-20%	-49%	-47%	-40%	-63%	-78%	-85%	-92%	-94%	-95%	-97%
Grade Control Structures	-33%	-17%	-43%	-36%	-33%	-54%	-66%	-49%	-45%	-56%	-66%	-81%
Grade Control Structures + LT1	-35%	-17%	-43%	-37%	-35%	-63%	-80%	-86%	-92%	-94%	-95%	-97%

Table 4.6 Comparison of Measures to Existing (No Action) Cumulative Output from the SRS and Sediment Load at the Mouth of the Toutle River by 2035 for the 95% Exceedance Forecasting Sequence

Measures	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Output from SRS by 2035 (Million Tons): 95% Exceedance											
Existing	106.7	47.5	22.8	10.6	8.1	6.5	4.7	1.3	0.7	1.2	1.5	1.8
Raised SRS	54.9	36.8	8.1	1.7	2.7	1.8	1.3	0.4	0.3	0.5	0.6	0.7
Raised SRS + LT1	--	--	--	--	--	--	--	--	--	--	--	--
Grade Control Structures	48.4	36.3	7.2	0.6	1.4	1.0	0.8	0.2	0.1	0.2	0.2	0.2
Grade Control Structures + LT1	--	--	--	--	--	--	--	--	--	--	--	--
	% Difference Relative to Existing											
Raised SRS	-49%	-23%	-64%	-84%	-67%	-72%	-71%	-68%	-59%	-60%	-60%	-61%
Grade Control Structures	-55%	-24%	-68%	-94%	-82%	-84%	-82%	-82%	-83%	-85%	-85%	-87%

Measrues	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Sediment Load at Mouth of Toutle River by 2035 (Million Tons): 95% Exceedance											
Existing	131.8	52.8	26.3	16.3	13.2	8.4	5.4	2.2	1.3	1.8	2.1	2.0
Raised SRS	80.0	42.0	11.5	7.5	7.8	3.7	2.1	1.3	0.9	1.1	1.2	1.0
Raised SRS + LT1	73.0	42.0	11.5	7.3	7.5	2.8	1.0	0.3	0.1	0.2	0.2	0.2
Grade Control Structures	73.4	41.6	10.6	6.4	6.5	2.9	1.6	1.1	0.7	0.8	0.8	0.5
Grade Control Structures + LT1	68.5	41.5	10.6	6.2	6.2	2.2	0.8	0.3	0.1	0.2	0.2	0.2
	% Difference Relative to Existing											
Raised SRS	-39%	-20%	-56%	-54%	-41%	-56%	-61%	-40%	-32%	-39%	-44%	-52%
Raised SRS + LT1	-45%	-20%	-56%	-55%	-43%	-66%	-82%	-88%	-90%	-91%	-91%	-92%
Grade Control Structures	-44%	-21%	-59%	-61%	-51%	-66%	-71%	-49%	-45%	-55%	-63%	-74%
Grade Control Structures + LT1	-48%	-21%	-60%	-62%	-53%	-73%	-84%	-88%	-90%	-91%	-91%	-92%

Table 4.7 Comparison of Measures to Existing (No Action) Cumulative Output from the SRS and Sediment Load at the Mouth of the Toutle River by 2035 for the Minimum Forecasting Sequence

Measure	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	63
	Cumulative Output from SRS by 2035 (Million Tons): Min Sequence											
Existing	74.4	30.4	14.9	8.0	6.7	4.9	3.3	1.0	0.7	1.2	1.5	1.8
Raised SRS	37.1	24.3	6.2	1.6	1.7	1.2	0.8	0.3	0.2	0.3	0.3	0.4
Raised SRS + LT1	--	--	--	--	--	--	--	--	--	--	--	--
Grade Control Structures	32.7	24.0	5.5	1.1	0.7	0.6	0.4	0.1	0.1	0.1	0.1	0.1
Grade Control Structures + LT1	--	--	--	--	--	--	--	--	--	--	--	--
	% Difference Relative to Existing											
Raised SRS	-50%	-20%	-59%	-80%	-75%	-75%	-74%	-75%	-77%	-78%	-78%	-79%
Grade Control Structures	-56%	-21%	-63%	-86%	-90%	-88%	-89%	-90%	-91%	-92%	-92%	-92%

Measure	Total	CM	VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG
		0.0625	0.125	0.25	0.5	1	2	4	8	16	32	64
	Cumulative Sediment Load at Mouth of Toutle River by 2035 (Million Tons): Min Sequence											
Existing	93.9	34.3	17.5	12.3	10.6	6.4	3.9	1.8	1.3	1.8	2.0	2.0
Raised SRS	56.5	28.2	8.7	5.9	5.6	2.7	1.5	1.0	0.7	0.8	0.8	0.6
Raised SRS + LT1	51.4	28.1	8.7	5.7	5.3	2.1	0.7	0.2	0.1	0.1	0.2	0.1
Grade Control Structures	52.1	27.9	8.0	5.4	4.6	2.0	1.0	0.9	0.6	0.7	0.6	0.4
Grade Control Structures + LT1	48.4	27.9	8.0	5.3	4.4	1.6	0.6	0.2	0.1	0.1	0.2	0.1
	% Difference Relative to Existing											
Raised SRS	-40%	-18%	-50%	-52%	-47%	-58%	-63%	-43%	-45%	-53%	-60%	-69%
Raised SRS + LT1	-45%	-18%	-50%	-53%	-50%	-68%	-82%	-88%	-91%	-92%	-92%	-93%
Grade Control Structures	-44%	-19%	-54%	-56%	-56%	-68%	-75%	-51%	-53%	-62%	-70%	-80%
Grade Control Structures + LT1	-48%	-19%	-54%	-57%	-59%	-75%	-86%	-88%	-91%	-92%	-92%	-93%

Figure 4.3 shows the cumulative sediment at the mouth of the Toutle River by 2035 for material that is < 0.125mm.

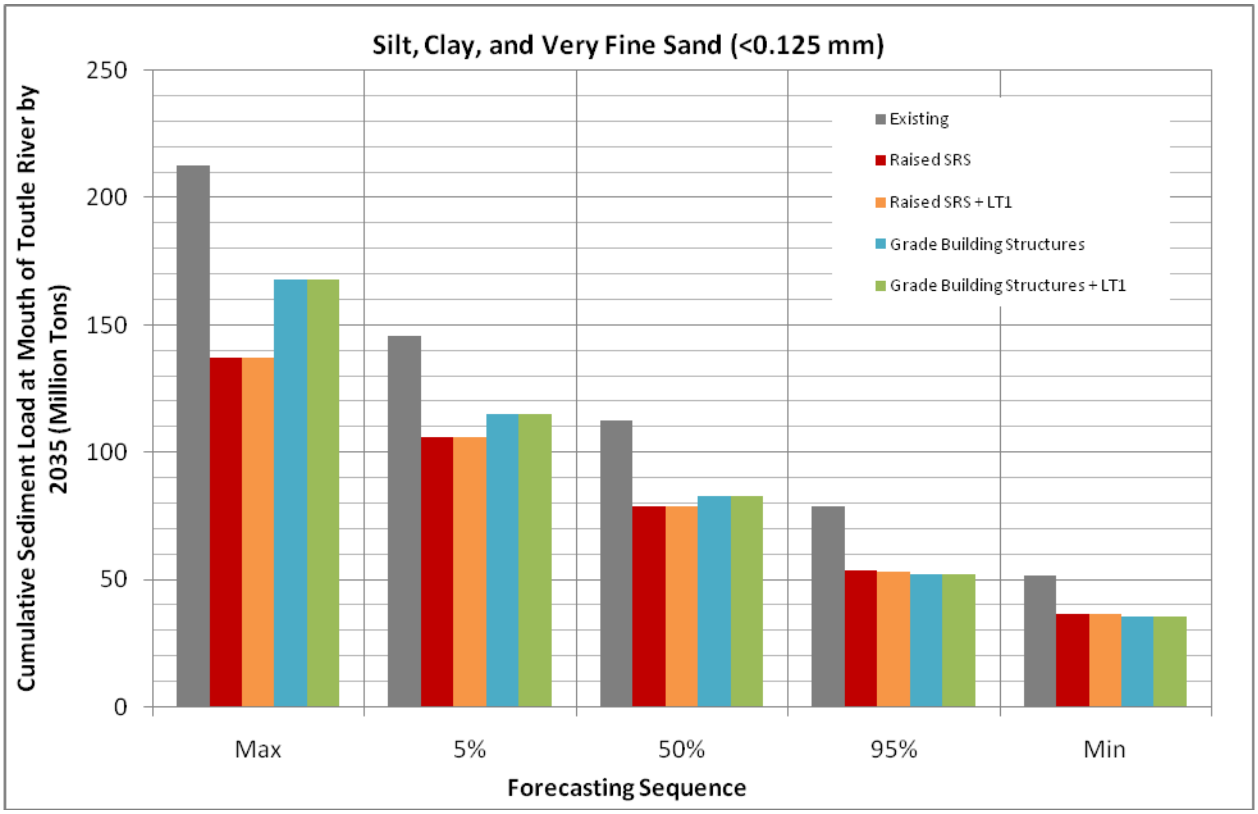


Figure 4.3 Cumulative Sediment Load at Mouth of Toutle River by 2035 for selected forecasting sequences for Silt, Clay, and Very Fine Sand.

Figure 4.4 compares the cumulative sediment load at the mouth of the Toutle River by 2035 for material between 0.125 mm and 2 mm.

For material between 0.125 and 2 mm the raised SRS measure decreases the cumulative sediment load at the mouth of the Toutle River by 47% to 53%. The raised SRS measure paired with the LT1 sump decreases the load from 51% to 58%, with the LT1 sump improving sediment reduction by only 4% to 5%. The grade control structures decreases the cumulative sediment load at the mouth of the Toutle River by 24% to 61%. With the addition of the LT1 sump the decrease in sediment load ranges from 28% to 65%, with an improvement of 4% to 6%.

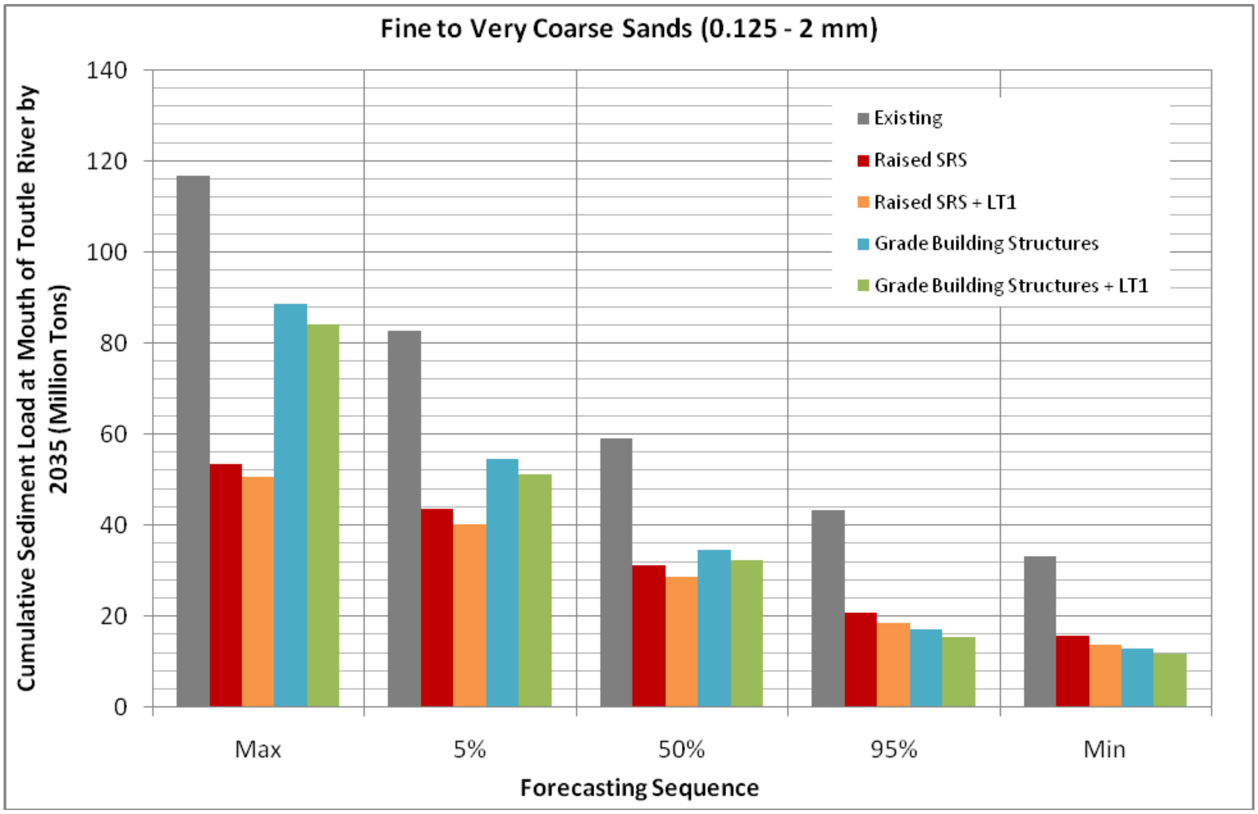


Figure 4.4 Cumulative Sediment Load at Mouth of Toutle River by 2035 for selected forecasting sequences for Fine to Very Coarse Sands (0.125 – 2 mm)

5.0 Recommendations

Each of the four measures has been compared with existing condition sediment yield at the mouth of the Toutle River: 1) Raised SRS, 2) Raised SRS + LT1 sump, 3) Grade Control Structures, and 4) Grade Control Structures + LT1 sump. Each of the measures contains the same degree of uncertainty as the existing condition sediment budget, and the analyses were solely comparisons. Uncertainty in the performance of each measure was not investigated. Based on the comparisons, the following recommendations are made:

- In each case the effect of the LT1 sump was small, in the range of 4% to 6%, relative to the raised SRS and grade control structures. The LT1 sump was found to be ineffective at trapping material < 1 mm. Level 2 analysis of the LT1 measure is not recommended unless the need emerges to remove gravel from the system.

- Raising the SRS is an option that is recommended for additional Level 2 analysis. The raised SRS is more effective than the Grade Control Structures for the 50% exceedance, 5% exceedance, and maximum sediment load forecasts. Longevity is a positive consideration for this measure, and cost and delay in implementation are negative considerations for this measure.

- Grade Control Structures is an option that is recommended for additional Level 2 analysis. A positive consideration for these structures is that implementation can occur rapidly. Lack of familiarity with this type of structure, limited capacity, and continued maintenance/construction until complete are negative considerations.

Lower Cowlitz Expanded Floodplain

Measure Description

The expanded floodplain measure decreases flood stages in the Lower Cowlitz River by restoring the natural floodplain terrace along portions of the lower 20 miles of the Cowlitz River. Levees and infrastructure are set back and dredge spoil and fill above the historic floodplain terrace is removed increasing conveyance during flood flows and lowering flood stages. The setback and excavated area would be managed as a flood protection measure and remain as managed greenspace. Figure EF1 shows the suite of activities that combine to make the expanded floodplain measure.

The activities shown in Figure EF1 represent an aggressive expansion of the floodplain to investigate the potential of the measure to reduce flood stages. The combined activities have a cumulative effect with downstream measures providing benefit for some distance upstream. The area proposed for floodplain expansion is largely privately owned with a mix of residential, commercial, industrial and agricultural uses. Floodplain expansion along the Longview and Kelso levees effects infrastructure most greatly involving relocation of levees, rail lines, roadways, as well as extension of two bridges and removal of dredge spoils in the setback area. Expansion along the Lexington levee alleviates the existing constriction at Rocky Point with a large setback of the Lexington levee, re-terracing the reclaimed floodplain and the extension of one bridge. Setback of the Castle Rock levee was not required to reduce flood stage due to the lack of geographic constraints on the opposite bank. Significant dredge spoil removal and the extension of one bridge comprise the activities in the vicinity of Castle Rock.

The setback areas would be re-terraced to inundate during events larger than the 50% to 20% annual exceedance probability (AEP) flood flows. Average annual sediment transport capacity is not expected to change with this measure as it does not modify the river below historic bank elevations. During more extreme flood events, silts and fine sands are abundantly supplied by the Toutle River and observed depositing in existing connected floodplain terraces along the lower Cowlitz River. Expansion of the floodplain would likely induce more deposition in the floodplain during these extreme events as average velocities in floodplains would be decreased. Aging floodplains would vegetate resulting in rougher overbanks and a further decrease in off-channel velocities causing additional off-channel deposition. It is expected that continued deposition in the expanded floodplain would raise the terrace and reduce the effectiveness of the measure without occasional maintenance of the created greenspace. This maintenance would include periodic removal of deposited soils, clearing of understory vegetation and thinning of trees.

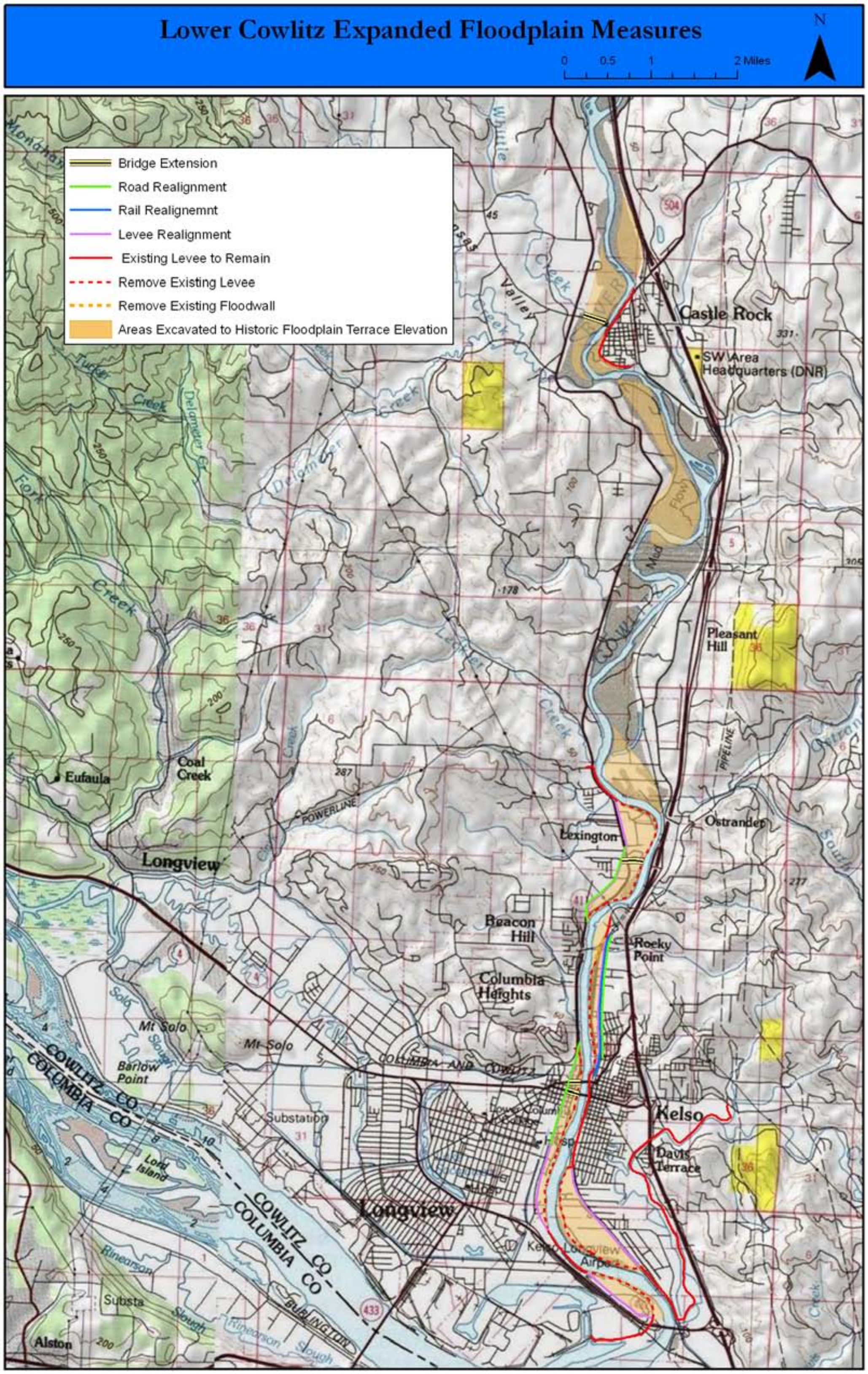


Figure EF1: Expanded Floodplain Measures

Analysis

Stage

An existing steady state HEC-RAS model of the lower Cowlitz extending from the Columbia River to the confluence with the Toutle River was utilized to investigate the effects of expanding the floodplain on flood flow stages. The model contained an existing condition geometry calibrated to flood events. The existing condition model geometry was then modified to reflect the expanded floodplain condition shown in figure EF1. Flows bounding the authorized level of protection (LOP) flows were run with both the existing condition and expanded floodplain. These bounding flows were the 1% and 0.5% AEP flows. Profiles for both conditions and both flows are shown in figure EF2.

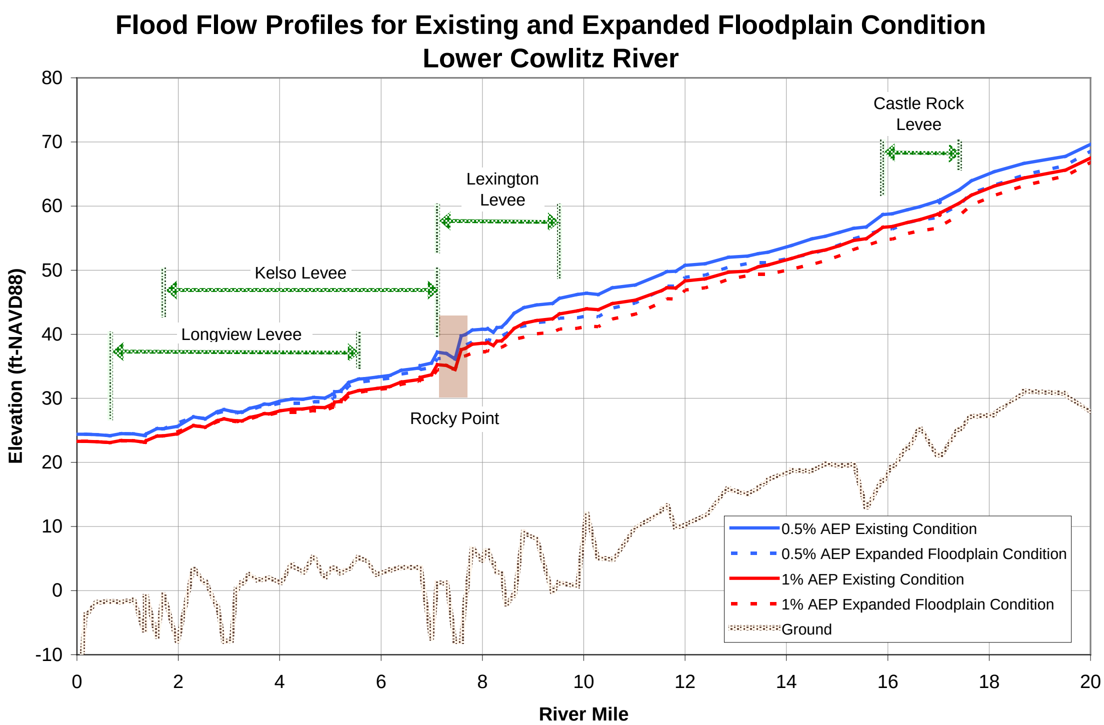


Figure EF2: Flood Flow Profiles along Lower Cowlitz River

	1% AEP Flow	0.5% AEP Flow
Longview Levee	0.2	0.2
Kelso Levee	0.2	0.4
Lexington Levee	1.4	1.9
Castle Rock Levee	1.9	2.3

Table EF1: Average Reduction in Stage Due to Expanded Floodplain Measures (ft).

The potential for stage reduction is limited along the Longview and Kelso Levees due to the fixed backwater elevation at the Columbia River. A relatively wide expansion of the floodplain between RM 1.7-4.2 yielded limited stage reduction benefits (0.2 ft average) along the Longview Levee and only marginally better benefits along the Kelso levee due to its extents further upstream.

The largest step in the existing condition backwater profile occurs at a restriction in the river created between the Lexington levee and the natural feature Rocky Point near river mile 7.5. Levee setback and re-terracing (removal of dredge spoils and natural fill above the 2-5 year flood stage) as shown in figure EF3 provides the largest opportunity for flood stage reduction along the reach. The potential for average flood stage reductions along the relocated Lexington Levee range from 1.4 to 1.9 ft in the LOP range of flows.

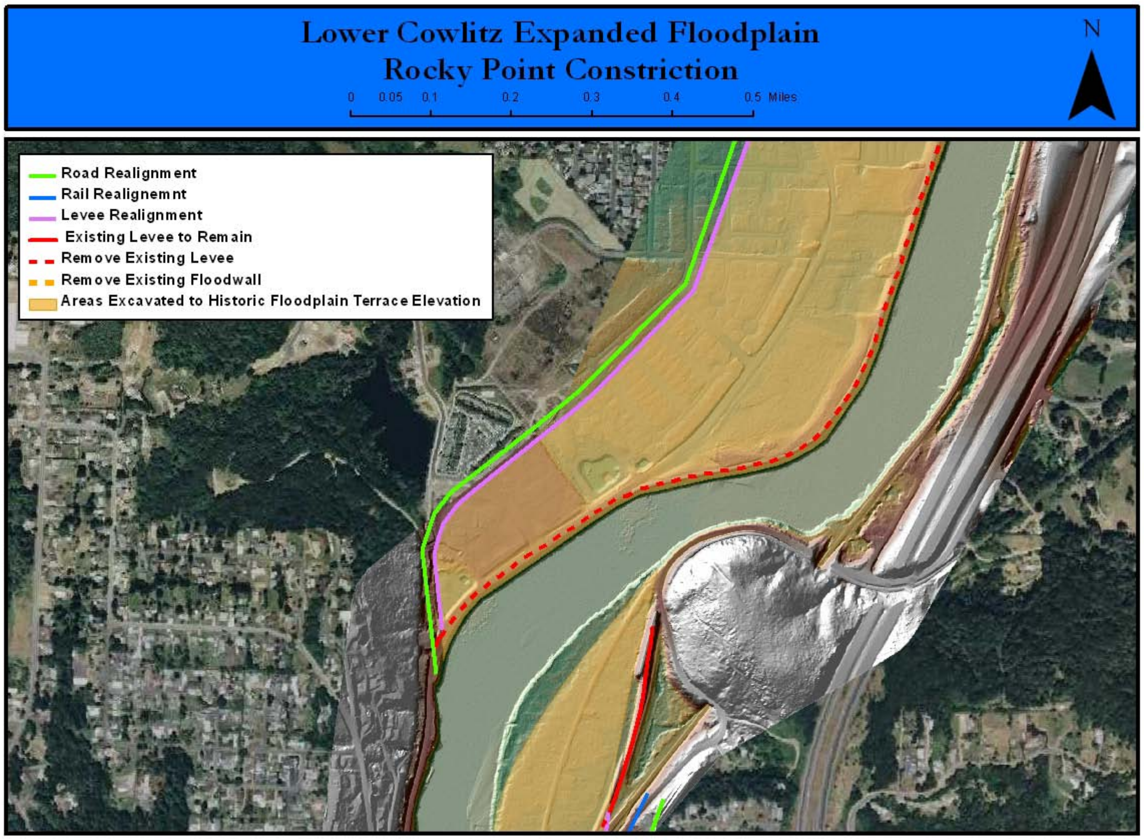


Figure EF3: Rocky Point Constriction

Extension of the stage reductions achieved at the Lexington levee upstream past the Castle Rock levee is largely accomplished by removal of dredge spoils in the historic floodplain and restoration of a terrace between the 2 and 5 yr flood event stages. The potential for average flood stage reductions along the Castle Rock levee range from 1.9 to 2.3 ft in the LOP range of flows.

Sedimentation

The existing condition and expanded floodplain HEC-RAS model geometries were converted to mobile bed HEC-RAS models with the addition of previously developed sediment bed properties, sediment load rating curves and two water years (2007-2008) of daily average upstream inflow and downstream stage data. The mobile bed model is uncalibrated and used as relative comparison only between the existing condition and the expanded floodplain condition. Water year 2007 represents a high discharge and sediment loading year and can be used to test the overbank deposition assumption. Figures EF5 and EF6 depict sediment deposition over the two year mobile bed run for the existing condition and expanded floodplain cross section at the Castle Rock levee respectively.

The expanded floodplain cross section (EF6) shows significant deposition on the created floodplain terrace. This trend persists throughout the reach supporting the assumption that the overbank terraces will fill over time and require maintenance.

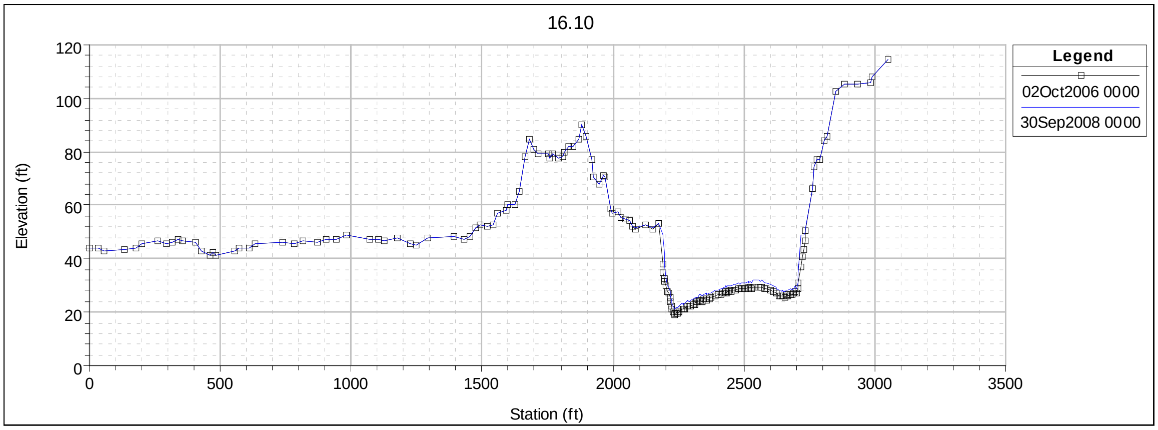


Figure EF5: Existing Condition Cross Section at Castle Rock Levee

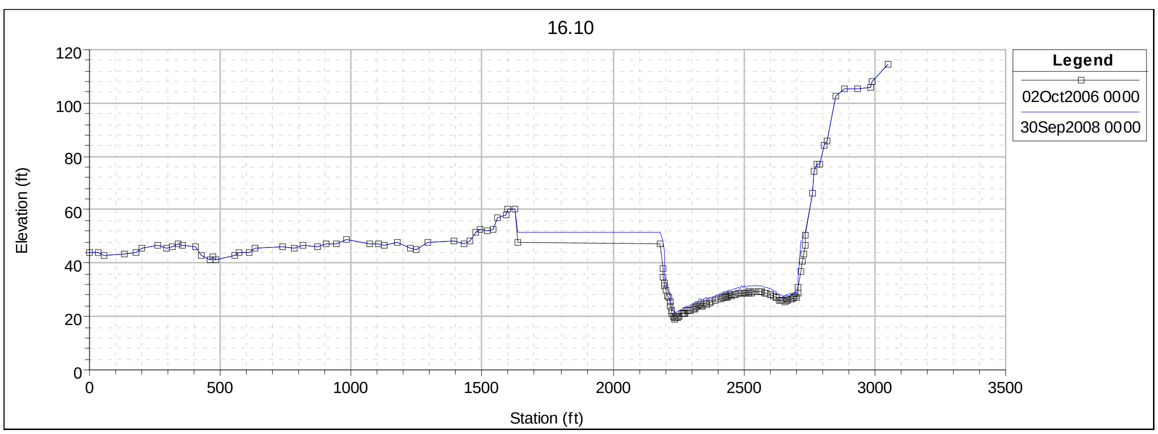


Figure EF6: Expanded Floodplain Cross Section at Castle Rock Levee

Conclusions

The lower Cowlitz expanded floodplain measure has limited ability to reduce flood stages in the LOP range of flows along the Longview and Kelso Levee.

Expansion of the floodplain at the constriction between the Lexington levee and Rocky Point has the greatest potential to reduce flood stages. A limited expansion of the floodplain at Rocky Point should be investigated in future phases as its stage reduction benefits would extend upstream along the Lexington levee.

Long term maintenance of the setback floodplain terraces including removal of deposited material and vegetation will be required for the measure to maintain its effectiveness.

Flushing Flows on the Lower Cowlitz

Measure Description

The flushing flow measure utilizes alternative regulation schemes at upstream flood control projects designed to flush sediment from the lower Cowlitz River with high flow pulses. The Cowlitz River at Castle Rock has been regulated by Mossyrock Dam (Riffe Lake) and Mayfield Dam (Mayfield Lake) since water year 1969 (FF1). These two reservoirs are part of the Cowlitz Project which is owned and operated by the City of Tacoma, Washington (Tacoma Power Company). Riffe Lake provides 360,000 acre-feet of flood control storage during December and January. Mayfield Lake acts as a re-regulating reservoir for releases from Mossyrock Dam. During the peak of the flood season (December and January), 360,000 acre-feet of flood control storage is available with the downstream flow objective of keeping the flow below 70,000 cfs at Castle Rock. A second maximum release objective limits releases below Mayfield Dam to 25,000 cfs to prevent flooding in communities along the Cowlitz between Mayfield Dam and Castle Rock.

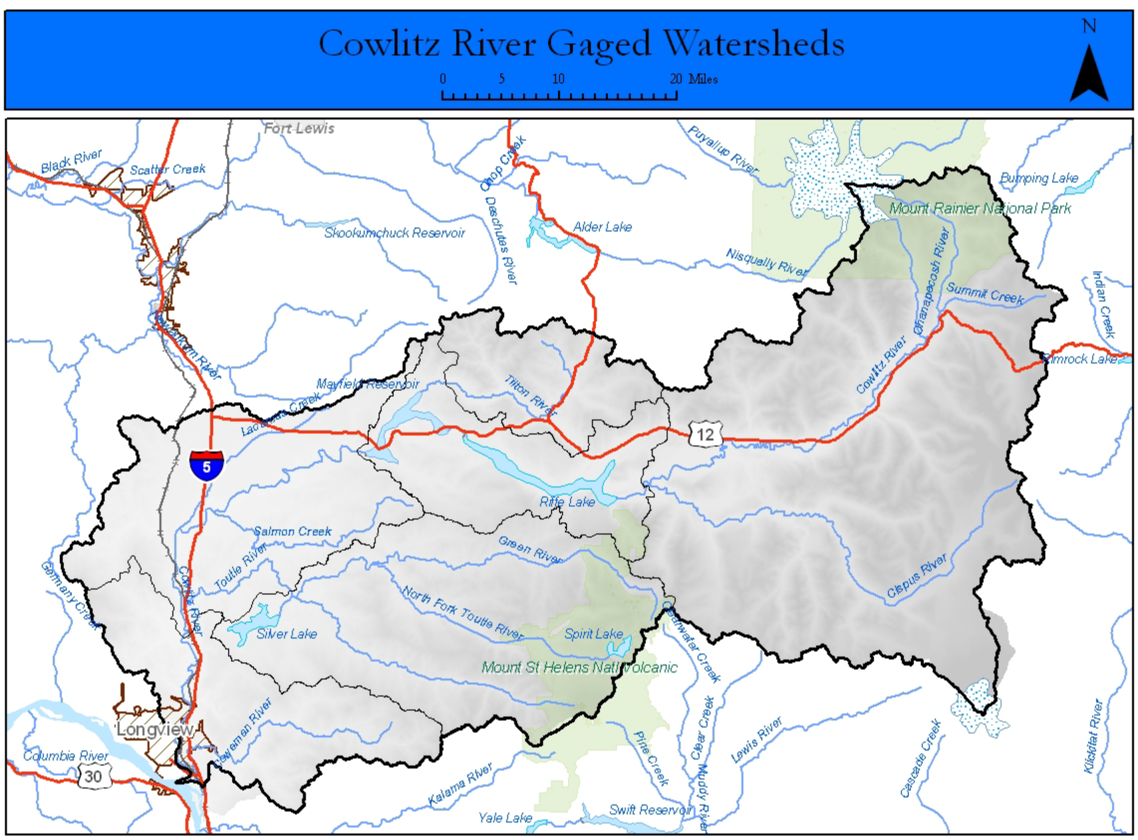


Figure FF1: Cowlitz Watershed and Regulation Projects

Two general flushing concepts are investigated as described below:

- Drawdown Flushing. Re-regulation of fall drawdown to winter flood control storage whereby water is evacuated from the pool prior to flood season with a higher pulse.

- Rain Event Flushing. Rain event re-regulation whereby water is released at a higher rate immediately after a large rain event.

For each general flushing concept, two re-regulation hydrographs are developed:

- 25 kcfs Max Release. This scheme releases a maximum of 25,000 cfs from Mayfield dam while not exceeding a maximum flow at Castle Rock of 50,000 cfs.

- 70 kcfs Control. This scheme regulates below a maximum flow of 70,000 cfs at the Castle Rock Gage and allows for releases from Mayfield in excess of the 25,000 cfs. This scheme is not feasible without development of additional flood protection projects on Cowlitz River but is informative concerning sensitivity of deposition related to regulated flows.

Analysis

An existing uncalibrated mobile bed HEC-RAS model of the lower Colwitz River was run for water years 2007 and 2008 with existing condition hydrology and four re-regulation inputs reflecting the concepts discussed in the measure description. The model runs are used to investigate the relative change in deposition in the Lower Cowlitz due to the flushing schemes. Figure FF2 shows the model geometry and boundary condition inputs required for mobile bed HEC-RAS. The only input modified for the flushing flow runs was the Cowlitz River Inflow.

Mobile bed HEC-RAS model is a quasi-unsteady state model and is not capable of modeling flow reversals or complete unsteady hydrodynamics. Additionally, run times for the model are prohibitively long when very high frequency boundary condition data is utilized therefore daily averaged data is used for all inputs. For these reasons, mobile bed results and relative trends in the lowest 5 miles of the Cowlitz model should be used with caution due to the tidal variation of the Columbia River and lowest portion of the Cowlitz.

Due to the uncalibrated mobile bed model, absolute values and rates of deposition have high uncertainly. All results should be compared back to the existing condition for a relative benefit

from the flushing flows. All model runs were highly depositional for the complete 2007 through 2008 water years due to the high observed sediment loadings in 2007. The relative comparison will be a percentage of change in deposition relative to the existing condition.

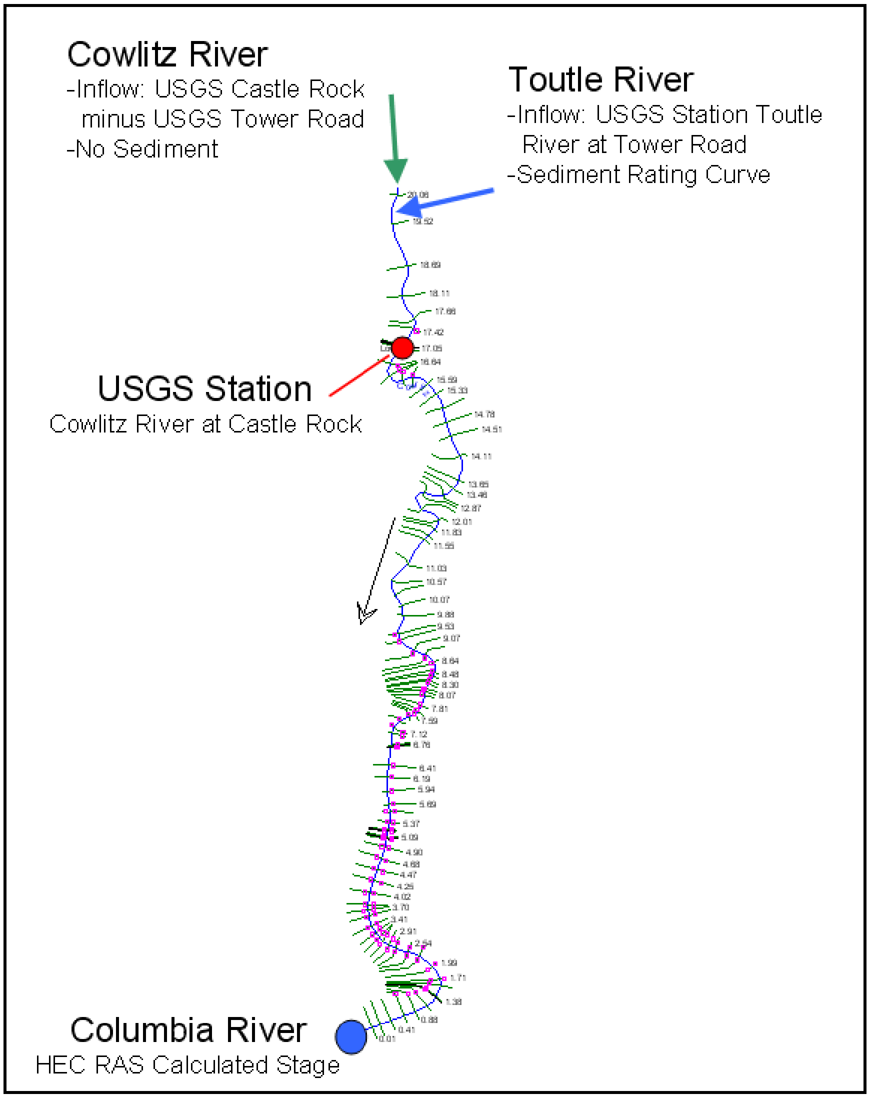


Figure FF2: Mobile Bed HEC-RAS Geometry and Inputs

Drawdown Flushing

In the fall of the year, up to 360,000 acre-feet of storage is evacuated to reach the winter flood control elevation by 1 December. These re-regulation schemes draw down the reservoir at an expedited rate creating a flushing pulse. Drawdown flushing is a scour inducing scheme as the pulse necessarily preempts the flood season and any pulse of sediment being introduced at the Toutle River. This requires the release to be large enough to induce movement of sediment from bed of the river and the duration to be long enough to transport sediment some distance downstream. Since the sediment source is the bed, the particles will necessarily entrain low in the water column and will be well positioned to settle back to the bed if stream power diminishes.

Figure FF3 shows the Cowlitz River inflows for the existing condition and two drawdown scenarios for water years 2007 through 2008. Volume is conserved for all three inflow hydrographs.

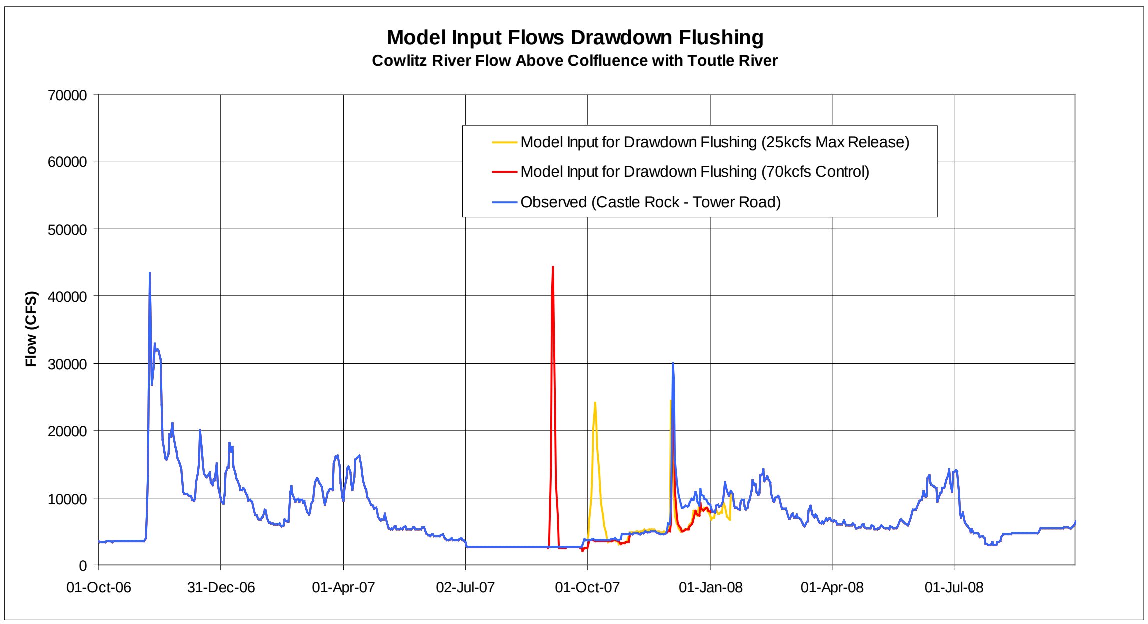


Figure FF3: Drawdown Flushing Event Cowlitz River Inflow HEC-RAS Inputs for Water Years 2007-2008

<u>25 kcfs Max Release Drawdown Flushing</u>

At this time there is not a restriction on the rate of change of outflow from Mossyrock/Mayfield Dams for flows greater than 6,000 cfs. For purposes of the analysis it was assumed that some rate of change on the outflow would be imposed on the flushing flow operation. The

assumption for increasing flow was not to increase outflow more than 50 percent in one day. The assumption for decreasing flow was not to decrease flow more than 20 percent in one day. These rates of change on the outflow similar to what Portland District uses at its projects.

Hydrology from the beginning of Water Year 2008 was used to model the flushing scenario. In the scenario flows would begin to ramp up starting on 1 October. Using the 50 percent rate of change restriction it would take about 7 days to reach a peak outflow of 24,000 cfs. After holding the peak outflow for one day, the flows were ramped down following the 20 percent outflow per day decrease restriction. The ramp down to 3,500 cfs, approximately the minimum October outflow, took about 10 days. The scenario resulted in a peak outflow from Mayfield of 24,500 cfs, and a peak flow at Castle Rock of 24,800 cfs.

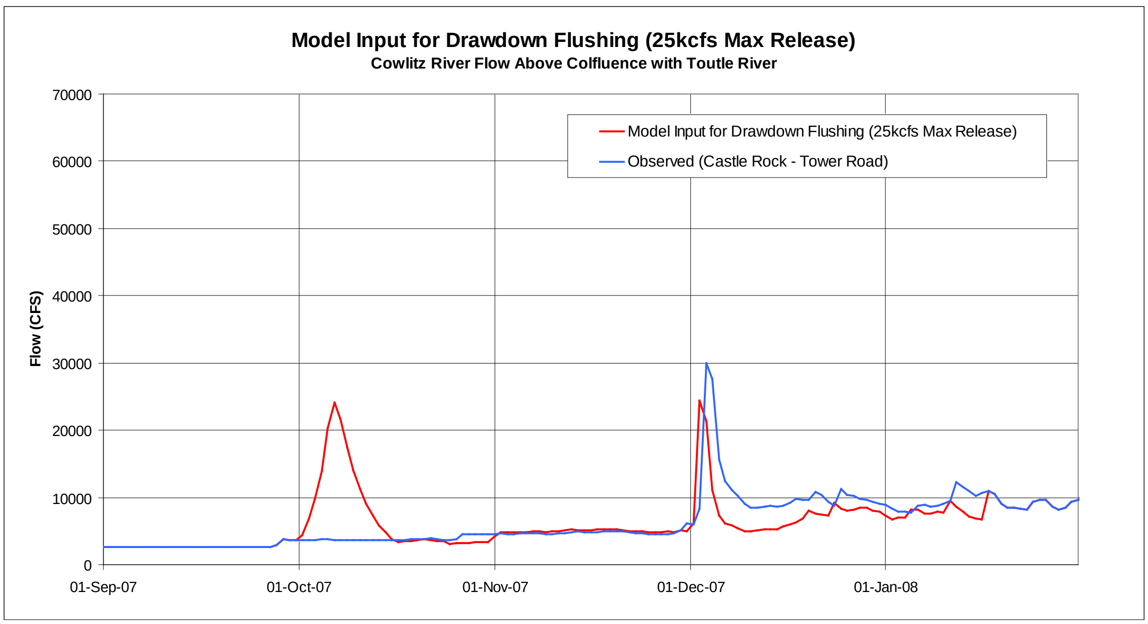


Figure FF4: Drawdown Pulse Cowlitz Inflow Hydrograph

<u>70 kcfs Control Drawdown Flushing</u>

The limiting factors on the fall drawdown 70 kcfs Control flushing scenario, in addition to the 70,000 cfs limit at Castle Rock, is the amount of storage available at Mossyrock. A high flow of 50,000 cfs could not be maintained for more than 24 hours given the storage that it would take to ramp up to that flow and then ramp back down after. For the purpose of this analysis it was assumed that the ramp up from a typical fall low flow of 1,500 to 2,000 cfs to 50,000 cfs would occur over 2 days. After 24 hours at 50,000 cfs the flow was ramped back down over 5 days.

Figure FF4 shows the zero volume change re-regulation of the drawdown with a pulse from the upper Cowlitz being sent in early September of 2007. Ramping limitation would affect the maximum flushing flow and duration.

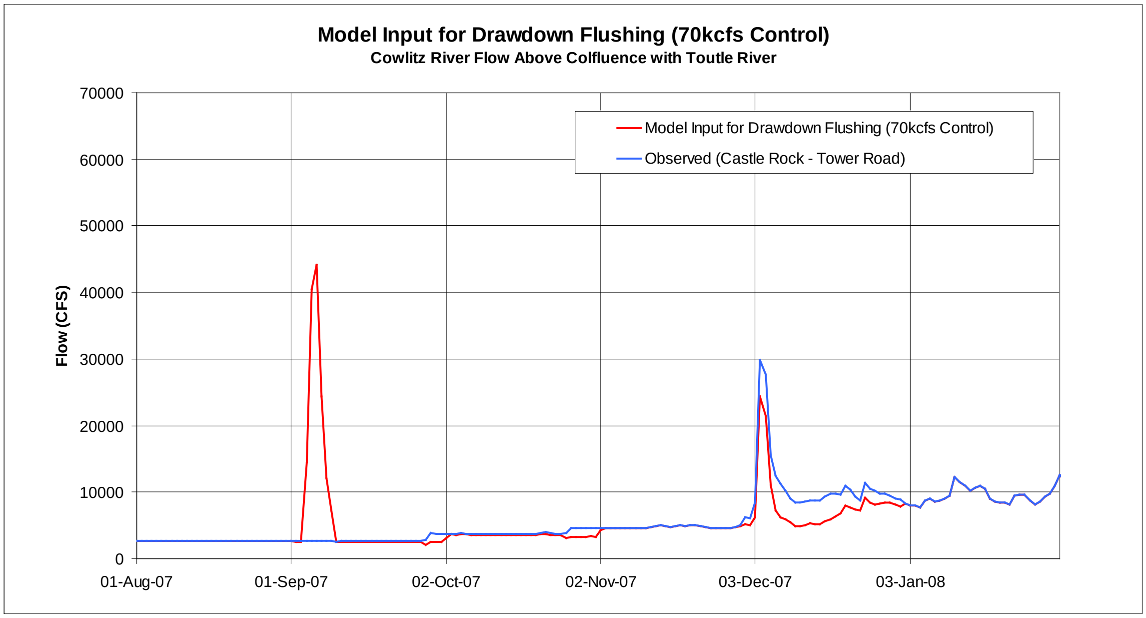


Figure FF5: Drawdown Pulse Cowlitz Inflow Hydrograph

Drawdown Flushing Results

Figures FF6 and FF7 show deposition rates (tons/mile) along the lower Cowlitz River for the existing condition and the drawdown flushing condition over the flushing period. Since September is a low flow period in Cascade streams, sediment inflow from the Toutle River results in negligible deposition in the existing condition. Both flushing pulse scenarios induces scour along the reach with the highest rates calculated in the lowest three miles (note that this is the tidal region where there is the least model confidence in trends). When the complete model run (2007 through 2008) is considered, the 25 kcfs Max Release drawdown flushing schemes reduced total deposition in the Lower Cowlitz by 3% while the 70 kcfs Control drawdown flushing scheme reduced total deposition by 15%.

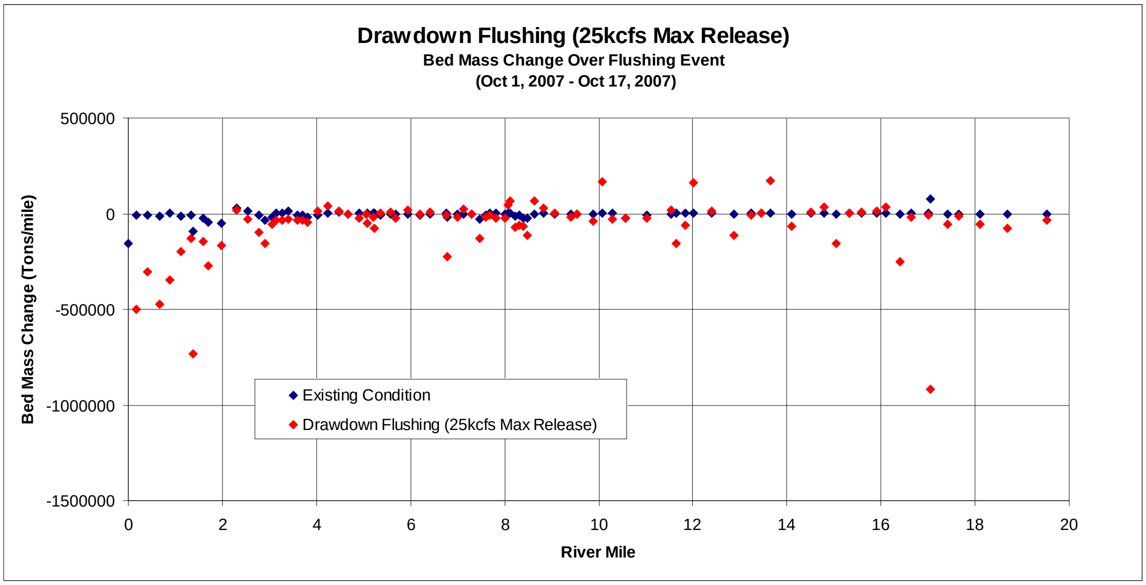


Figure FF6: Deposition Rates in the Lower Cowlitz River for the Drawdown Pulse, Oct 1, 2007 through Oct 17, 2007

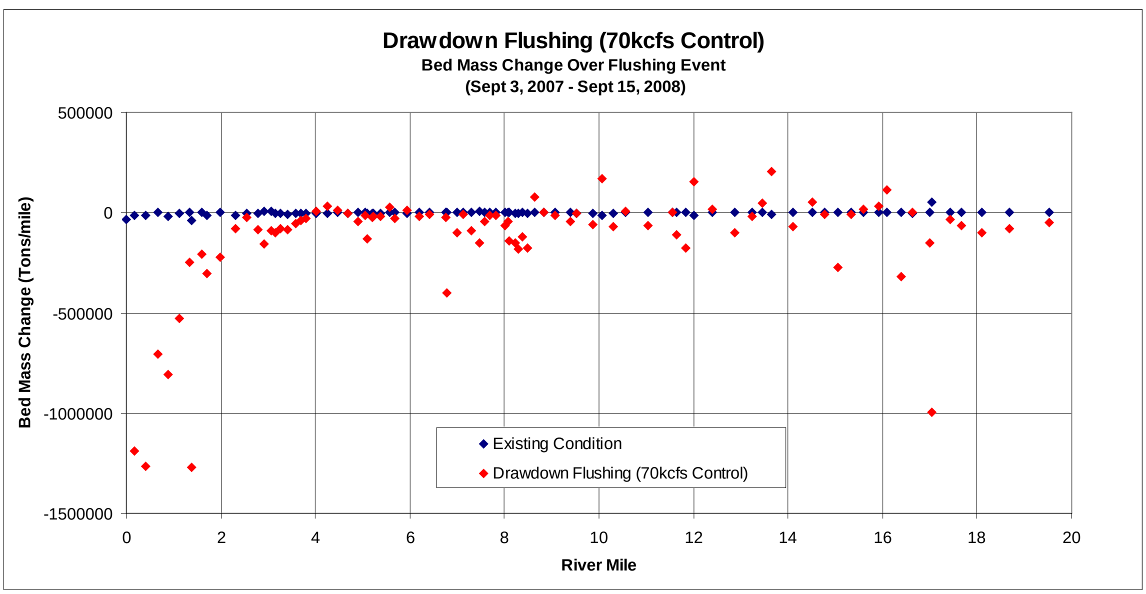


Figure FF7: Deposition Rates in the Lower Cowlitz River for the Drawdown Pulse, Sept 3. 2007 through Sept 15, 2007

Rain Event Flushing

The Rain Event Flushing scheme provides additional water to the Lower Cowlitz River when the Toutle River is producing a higher sediment load. Rain event flush can be scour inducing as well as deposition reducing. When highly sediment laden flows from the Toutle reach the Cowlitz, additional water provided from the upper Cowlitz flood control projects increases the transport capacity of lower Cowlitz reach flushing sediment through the system before it can settle on the bed. Large sediment carrying peak flows on the Toutle rarely extend more than a day or two in duration. By maintaining high flow from the upper Cowlitz, the lower Cowlitz maintains high transport capacity and reduces deposition in the Lower Cowlitz until the Toutle recession passes and sediment loads diminish. If high flows from the upper Cowlitz persist past Toutle recession, scour may occur in the Lower Cowlitz. If a storm is centered over the upper Cowlitz basin and does not greatly affect the Toutle, the high flows from the upper Cowlitz may act to scour the lower Cowlitz as transport capacities exceed the supply of sediment from the Toutle. Generally the rain event schemes moves the regulated hydrograph toward the natural unregulated flows.

Figure FF8 shows the Cowlitz River inflows for the existing condition and two rain event flushing scenarios for water years 2007 through 2008. Volume is conserved for all three inflow hydrographs.

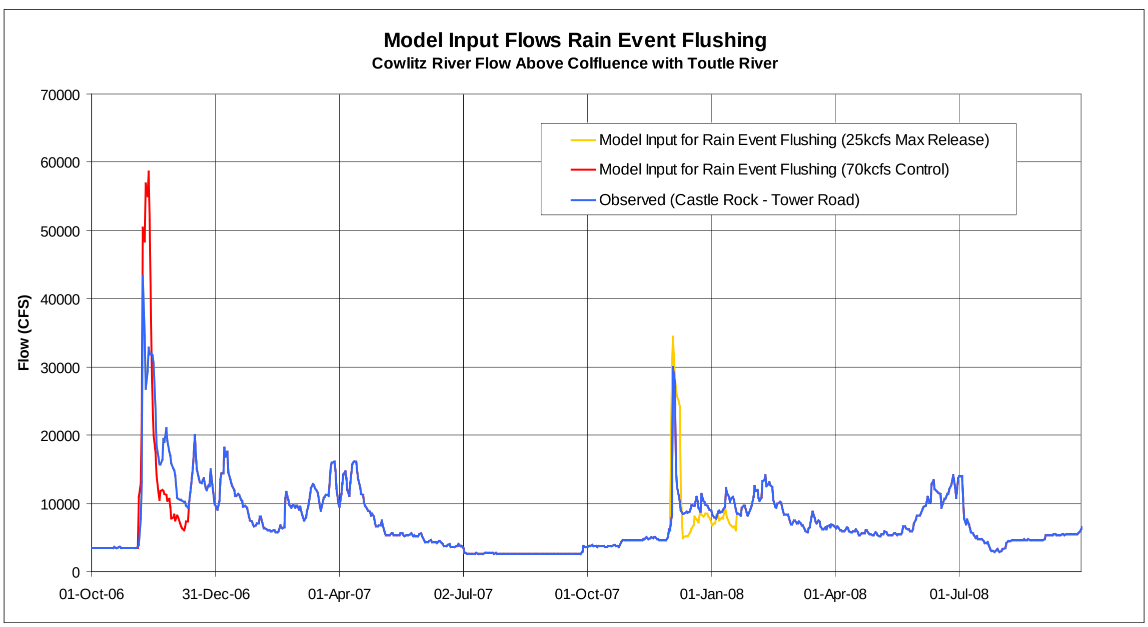


Figure FF8: Rain Event Flushing Scenarios Cowlitz River Inflow HEC-RAS Inputs for Water Years 2007-2008

25 kcfs Max Release Rain Event Flushing

The 25 kcfs Max Release scheme proposes to utilize a trigger flow in the Toutle River system to initiate a flow release from Mossyrock Dam. When the Toutle River at Tower exceeds a threshold flow, a maximum allowable release from Mossyrock would commence for a period of 5 days. Maximum releases are assumed to be 25,000 cfs from Mayfield dam while not exceeding a maximum flow at Castle Rock of 50,000 cfs. In water years 2007 and 2008 the December 2007 rain event was selected for the 25 kcfs Max Release Scenario. Re-regulation hydrographs are shown in figure FF9 along with the Toutle River at Tower Road flow that would initiate a release.

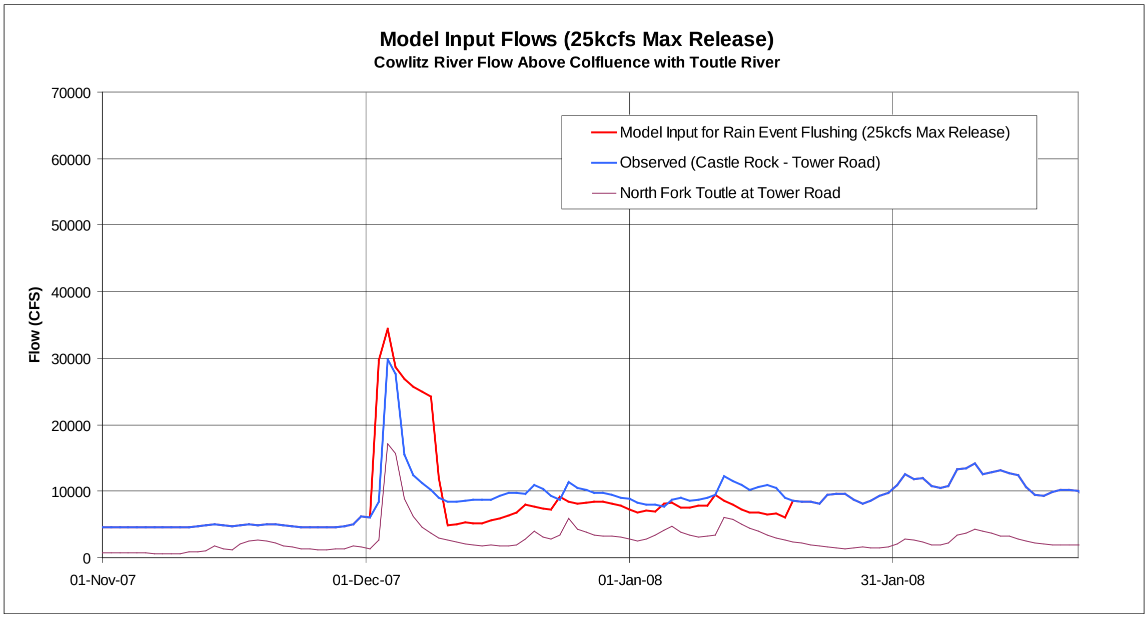


Figure FF9: Rain Event Flushing Cowlitz Inflow Hydrograph

70 kcfs Control Rain Event Flushing

The 70 kcfs Control rain event flushing scheme re-regulates the November 2006 storm to evacuate the reservoir after the flood peak has passed as quickly as possible while maintaining the flow at Castle Rock less than 70,000 cfs. The rain event was re-regulated in this manner resulting in 4 days near the target flow of 70,000 cfs at Castle Rock. The November 2006 event is a relatively large event in terms of peak discharge and volume. The unregulated 2 to 4 day volume upstream of the Riffe Lake was approximately 4 to 5% AEP.

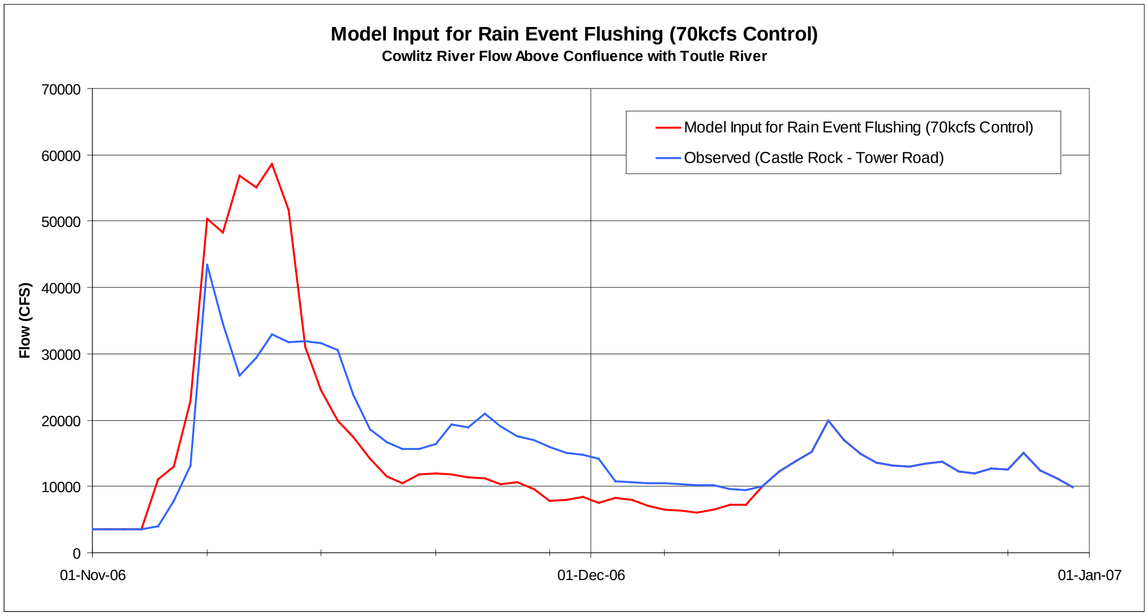


Figure FF10: Rain Event Flushing (70 kcfs Control) Cowlitz Inflow Hydrograph

Rain Event Flushing Results

Figures F11 and F12 show deposition rates (tons/mile) along the lower Cowlitz River for the existing condition and the rain event flushing condition over the re-regulation period. The 25 kcfs Max Release scheme decreased deposition between river miles 6 through 10 while the effect of the 70 kcfs Control flushing was to diminish deposition along the entire reach. When the complete model run (2007 through 2008) is considered, the 25 kcfs Max Release rain event flushing scheme reduced total deposition in the Lower Cowlitz by 12% while the 70 kcfs Control scheme reduced deposition by 30%.

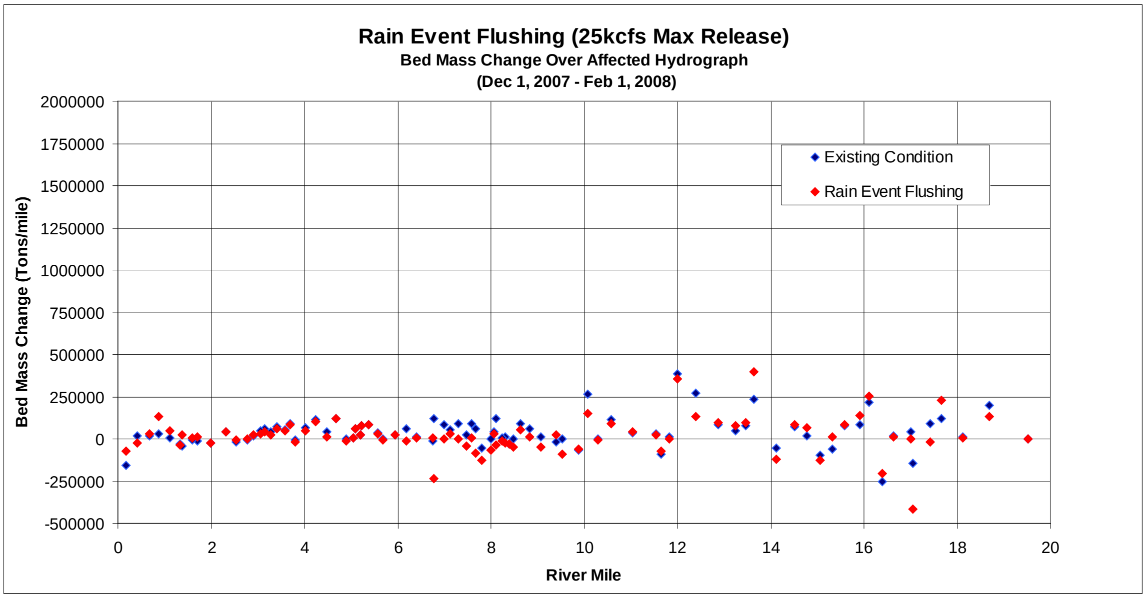


Figure FF11: Deposition Rates in the Lower Cowlitz River for the Rain Event Flushing (25kcfs Max Release), Dec 1, 2007 through Feb 1, 2009

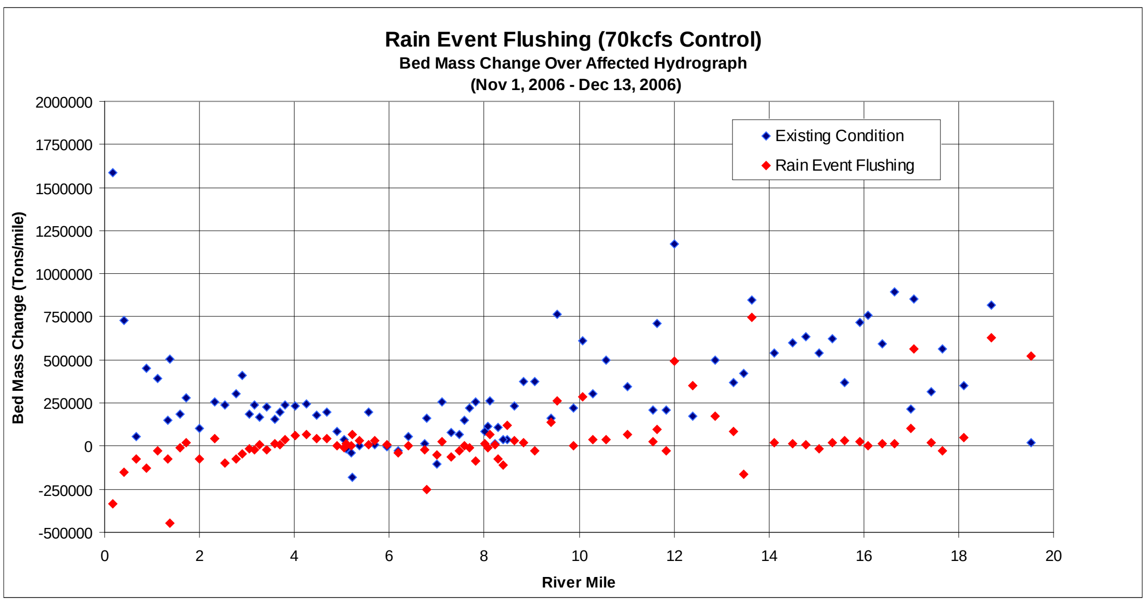


Figure FF12: Deposition Rates in the Lower Cowlitz River for the Rain Event Flushing (70 kcfs Control), Nov 1, 2006 through Dec 13, 2007

Conclusions

Re-regulation of flood control projects on the Cowlitz River can result in decreased deposition in the Lower Cowlitz River. Existing maximum release limitations in place due to flooding the Cowlitz River between Mayfield Dam and Castle Rock reduce the potential for flushing considerably. With current limitations, the drawdown pulse results in a marginal decrease in deposition. A greater potential for moving sediment lies in re-regulation of large storm events in the upper Cowlitz basin. Model results indicate that deposition in the Lower Cowlitz could be reduced by as much as 12% on a biannual basis if a flow release from the regulation projects is triggered by a sizeable storm on the Toutle.

Pile Dike Model Summary Report

Executive Summary

The Portland District of the U.S. Army Corps of Engineers is evaluating measures to manage sediment deposition downstream from Mount Saint Helens. As part of this effort, an initial study was launched using a 2-dimensional model (MIKE21-C) to evaluate the impact that a dike field would have on sediment transport within the lower reaches of the Cowlitz River.

Two fully coupled 2-dimensional hydrodynamic models were created of the lower 4.5 miles of the Cowlitz River: one of the existing channel and one with a series of 36 dikes placed throughout the lower portions of the river. Two six month Cowlitz River hydrographs representing high flow and typical flow water years for the Cowlitz River were run through both models to evaluate the effectiveness of the dike field in encouraging sediment movement through the Lower Cowlitz River.

The study area was discretized into four reaches that were compared over two years of flow within the 1992 and 1994 water year Cowlitz flow hydrographs. Sediment deposition and scour volumes were compiled and compared for the existing river configuration versus the proposed dike scenario.

Preliminary results indicate that at low flow the dike field performs similar to the existing condition, peak flow periods of typical Cowlitz flow years can transport up to 150% of the sediment compared to the existing condition, and at high Cowlitz River flows the dikes can increase sediment transport by two to three times through the system down to the mouth of the Cowlitz River.

Modeling Approach

The numeric model MIKE21-C (DHI Software) was used for the depth averaged hydrodynamic simulations. MIKE21-C is a two dimensional, depth averaged hydraulic model well suited to modeling water and sediment transport through sandbed rivers. The hydrodynamic module simulates water surface level and lateral and longitudinal velocity variations in response to a variety of forcing functions, including upstream Cowlitz River flow volume, tributary Coweeman River inflow, downstream Columbia River water surface elevation (which is a function of tide and incoming Columbia River flow in this system), bottom shear stress, and other possible influences including wind shear, barometric pressure, Coriolis acceleration, momentum dispersion, sources and sinks, evaporation, flooding and drying, and wave radiation stresses. Since the point of this

study was to evaluate the effect of a change in bed geometry (existing channel vs. existing channel with a dike field), wind shear, barometric pressure variation, evaporation, and wave radiation stresses were omitted.

Model Grid

MIKE21-C operates exclusively in SI units and is based on a curvilinear grid. A curvilinear grid is similar to a structured grid in that each cell has four sides, however, the cells can be non-orthogonal. The grid for the Lower Cowlitz River Study includes the lower 4.5 miles of the Cowlitz River and 6 miles of the Columbia River (1.5 miles downstream and 4.5 miles upstream from the Cowlitz including Carol's Channel). The 97,950 cell grid (653 cells in the Cowlitz River direction x 150 cells in the Columbia River direction) is shown in **Figure 1**. A small section of the model mesh is shown at an exaggerated scale (inset) to illustrate the density and orientation of the 2-dimensional grid layout.

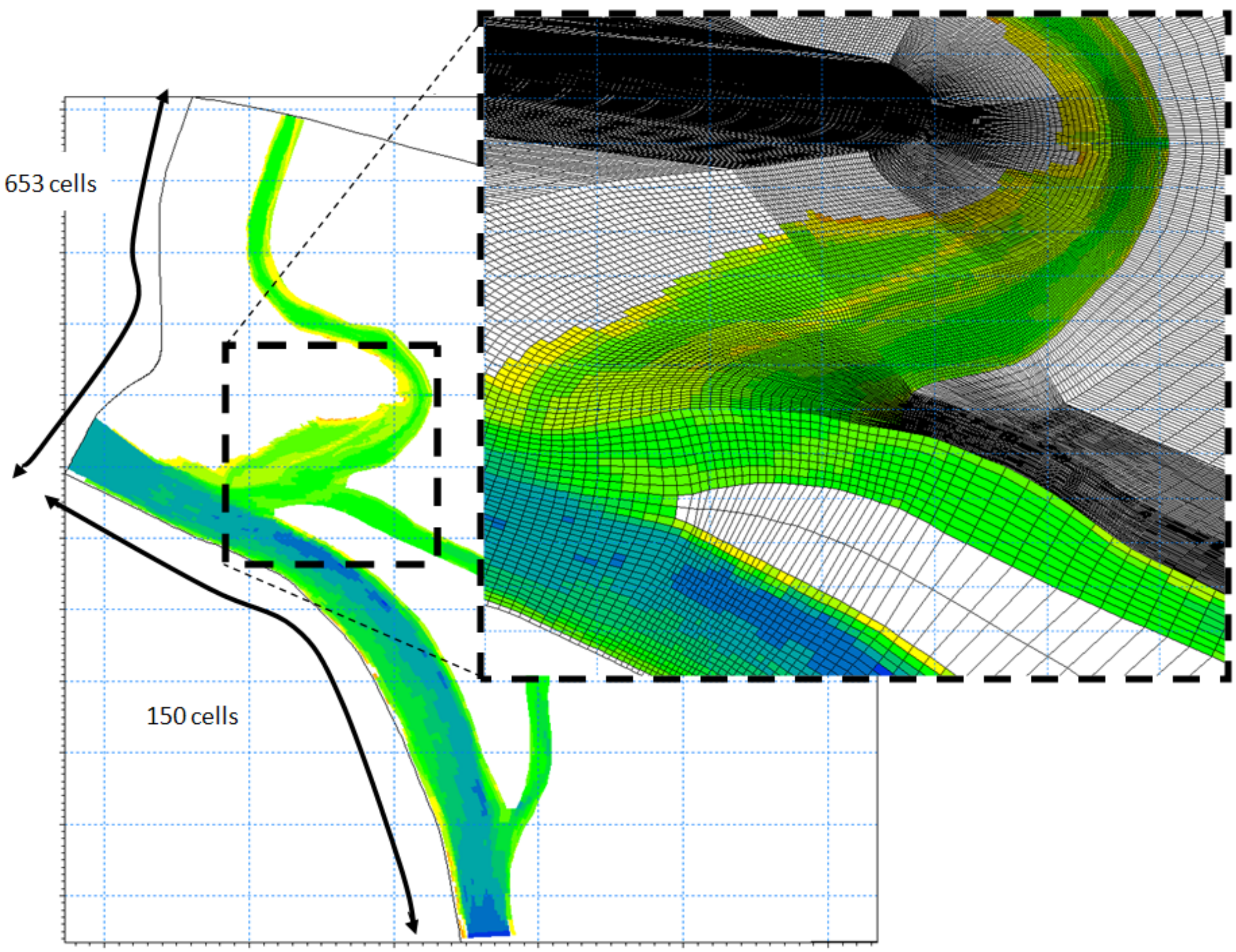


Figure 1. Lower Cowlitz Model Mesh

The resolution of the grid cells in the main flow channel of the Cowlitz river is approximately 10 meters by 10 meters (33 x 33 ft). This level of detail was necessary to allow incorporation of dikes into the model. The large number of cells (almost 100,000 cells) within the grid requires about 2 days of computer time to run a 190 day hydrograph (one year above baseflow) with a one second hydraulic time step.

Bathymetry

The model bathymetry (representing the river bed or physical channel geometry) was developed from USACE cross sectional surveys which were provided in Washington South State Plain feet NAD 1983 (NAVD 88 vertical datum) units. Channel bed data was interpolated between cross sections using the M21C Grid Generator interpolation routine. The bathymetry was converted to metric units by multiplying feet by 0.3048 in X (east), Y (north), and Z (elevation) dimensions. By emphasizing topographic detail in the direction of Cowlitz River flow, a smooth interpolated channel was created. High land elevations values of 6 meters were assigned to areas outside the channel to reduce the number of potential wet cells within the grid and accelerate computation times. **Figure 2** shows the Lower Cowlitz baseline bathymetry.

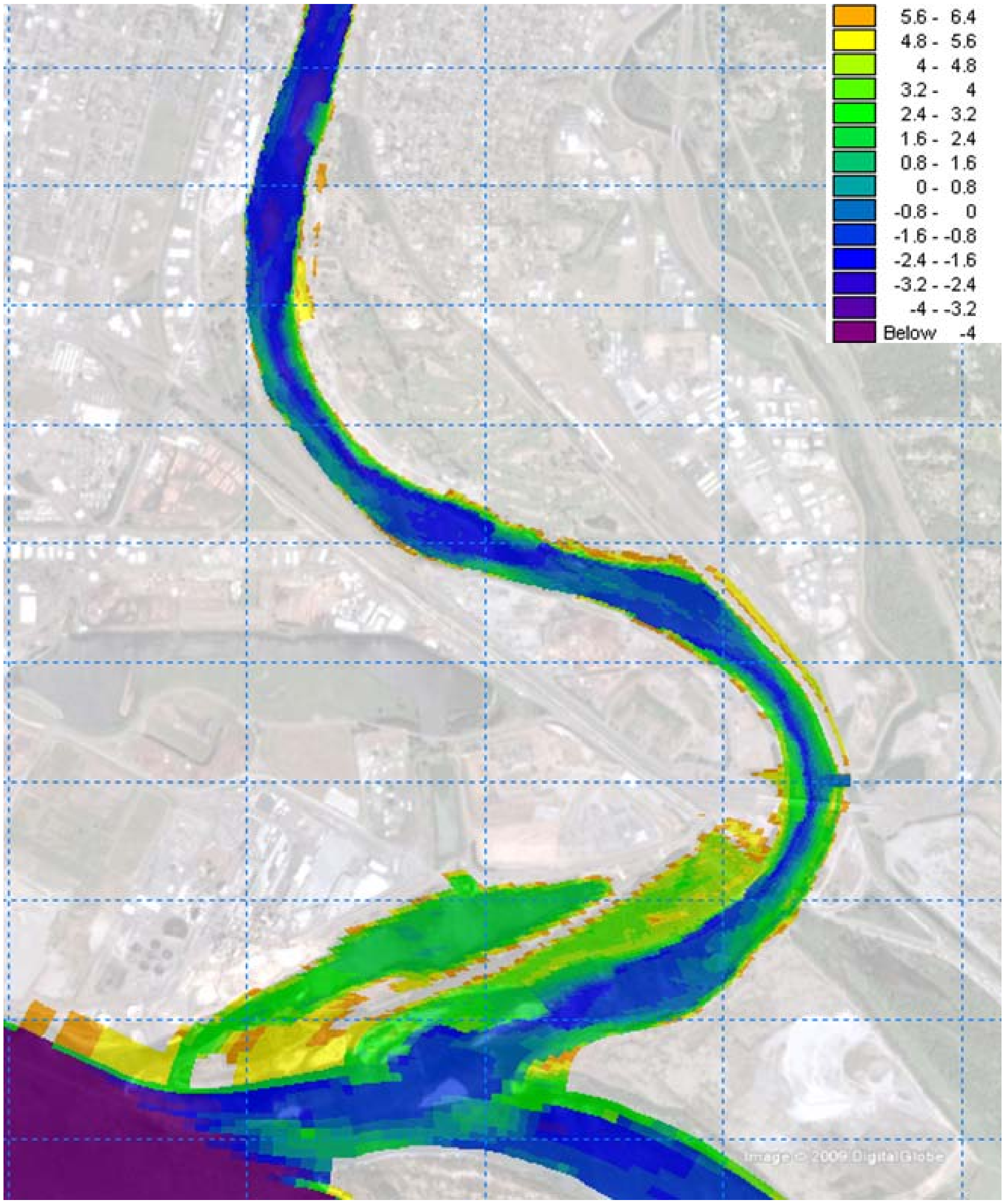


Figure 2. Lower Cowlitz Baseline Bathymetry (color coded elevations are in meters)

The proposed bathymetry is the same as the existing bathymetry with the addition of the dikes. Dikes are typically used as tools to improve the local sediment transport capacity of the main channel, thereby minimizing the need for maintenance dredging. Dikes are designed to convert a wide shallow channel to a deeper channel (which is more efficient for transporting sediment). The hydraulic effects of the dikes are most noticeable with stages at or below the top of the dike. As the stage continues to increase, the relative impacts of the dikes are diminished, particularly at overbank conditions. A key aspect of dike design is to balance the need for increased

sediment transport capacity at the low to intermediate stages with the requirement that flood stages will not be increased. For this reason the top elevation of dikes are generally constructed well below the top bank elevation (typically less than 1/2 to 2/3 of the bank height) to insure that their hydraulic impacts are negligible at the higher flows. Therefore, a properly designed dike system will result in lower stages at lower flows, with minimal changes in stage at the higher flows.

At approximately 30 locations throughout the study reach, dikes were added by raising mesh cells to a level of 2.3 meters in the upper 3 reaches and 1.5 meters in the lower reach (approximately the 50% exceedence discharge water surface elevation) so that they would train flow into a smaller active channel. The MIKE21-C model with dikes is shown in **Figure 3** (the dikes are shown in black). The upstream dikes are about 300 feet in length effectively constricting flow to about half of the original channel width. The initial dike placement was intended to illustrate the effect of dikes on sediment movement. Further study is necessary to refine dike locations, maximizing their ability to concentrate and mobilize sediment while minimizing impact on flood water elevation.

Hydrodynamic Simulation Period

River data for the Cowlitz (flow), Coweeman (flow), Columbia (flow and downstream stage) are all necessary as inputs for the 2-dimensional flow model. An overlapping period of record with hourly Columbia River flow and downstream water surface elevation (hourly data is necessary to account for tidal influences within the Columbia River), and mean daily Cowlitz and Coweeman River flow values was available between water years 1990 and 1996. Cowlitz sediment inflow values (in cubic meters per second for 6 size fractions ranging from 0.04 mm to 1.41 mm) was developed from available Cowlitz River sediment sample data for this period. A high flow period and a typical flow period on the Cowlitz River were selected to investigate the dike impacts. The time series period from September 20, 1991 to April 1, 1992 is a high Cowlitz River flow period and from October 1, 1993 to April 1, 1994 is a typical flow period. These two water years (1992 and 1994) within he available hydrologic records (**Figure 4**), were selected for the purpose of this study.

Model stability is related to time step length and grid cell size. High cell resolution (smaller cells) and high flow velocities requires the use of smaller time steps. The Lower Cowlitz River model was found to be stable with a hydraulic time step on the order of one second. A one second time step keeps the Courant Number (V_c) less than 0.20 when velocities (u) are less than 2 meters/second, and cell size (Δ x) is about 10 meters in the flow direction.

Equation 1 Courant Number

$$V_c = \frac{u \cdot \Delta t}{\Delta x}$$

The sediment time step was set at two minutes so that every 120 hydraulic time steps lead to one sediment transport update and bed recalculation.

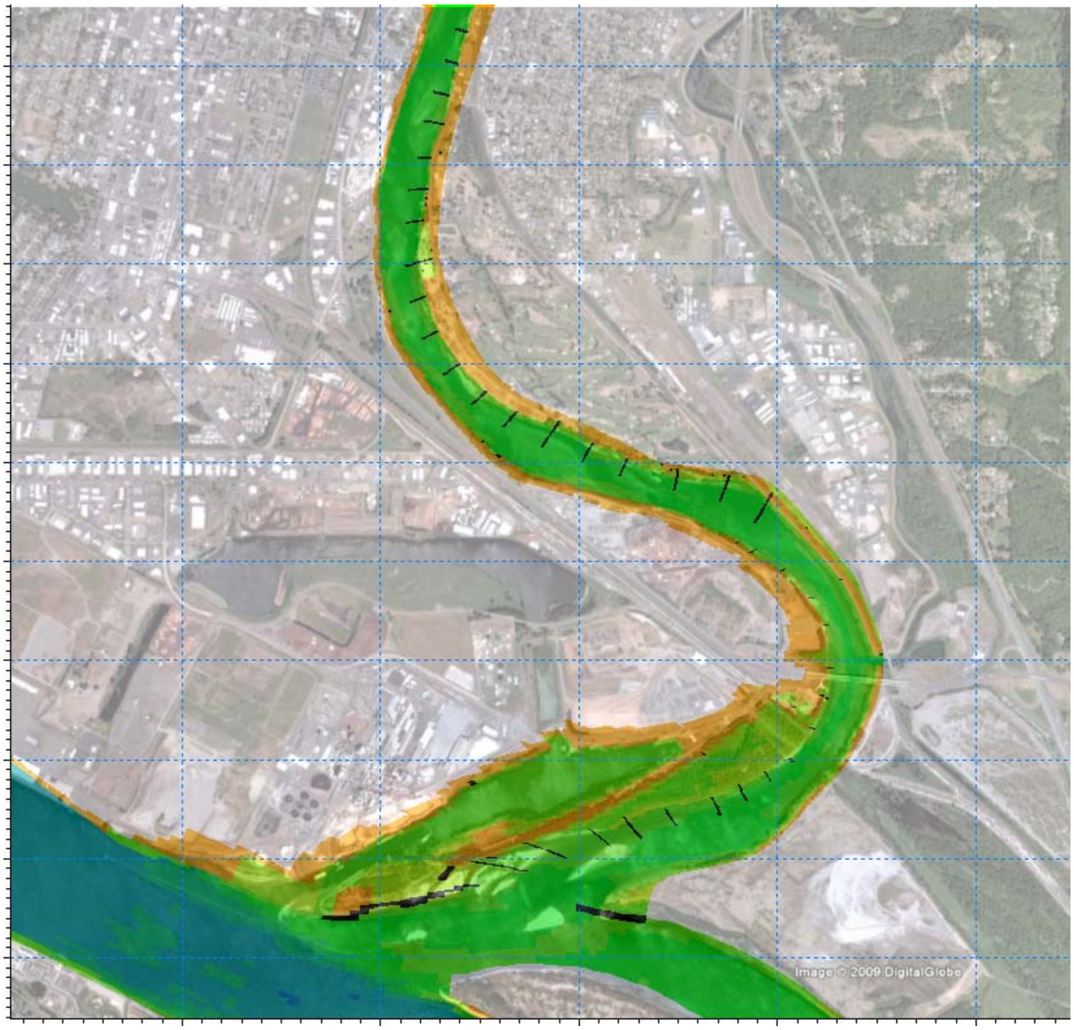


Figure 3. Dikes added to the Lower Cowlitz Model

Model Boundary Definitions

At each model boundary, either a water surface elevation or a flow is specified. Models must include at least one boundary where water surface elevation is defined and one boundary where flow is given. The remaining boundaries can specify water level or flow. This model has 4 model boundaries: the starting water surface elevation in the model is defined on the Columbia River

about 1.5 miles downstream from the Cowlitz confluence, incoming flow from the Columbia River is input approximately 4.5 miles upstream from the Cowlitz confluence, Cowlitz inflow and incoming sediment is defined at a boundary 4.5 miles upstream along the Cowlitz River, and Coweeman River inflow is entered upstream from the Highway 432 Bridge. The boundary condition input for hourly Columbia River water surface elevation is shown in **Figures 5** and **6** for water years 1992 and 1994 respectively. Hourly inflow from the Columbia, and mean daily inflow from the Cowlitz, and Coweeman Rivers is shown in **Figures 7** and **8** for 1992 and 1994. Sediment inflow rating curves by size fraction for the two periods are shown in **Figures 9** and **10**.

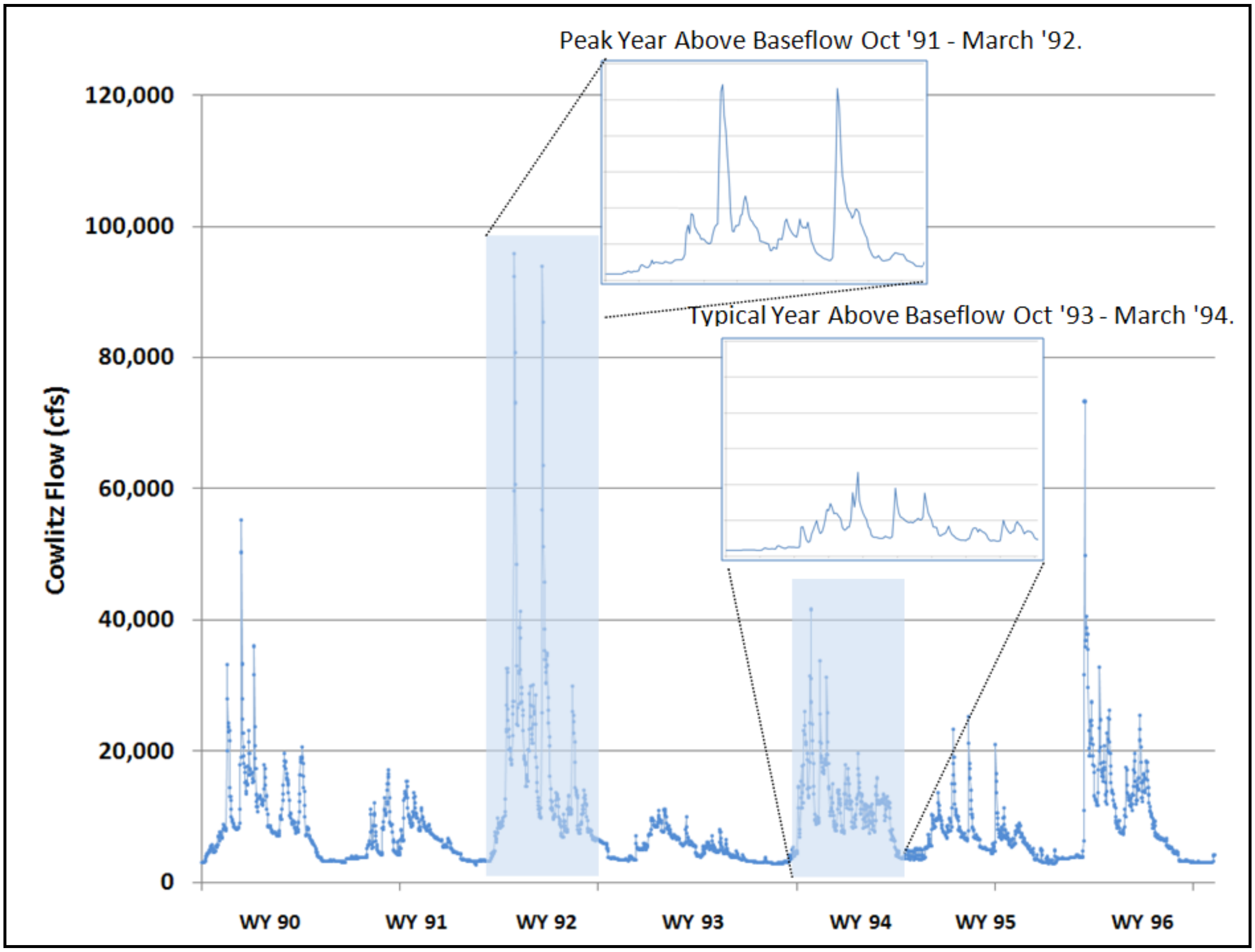


Figure 4. Cowlitz Flow (Water Years 1990 to 1996)

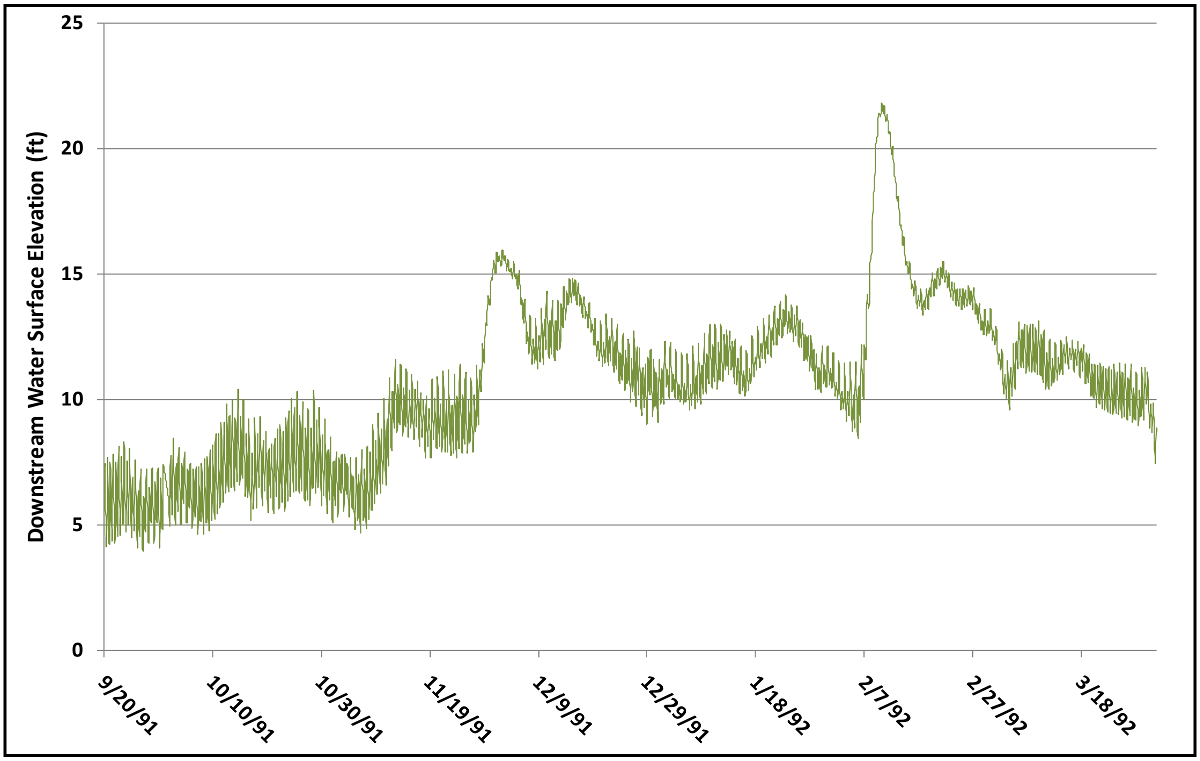


Figure 5. Boundary Condition Columbia River Water Surface Elevation - WY 1992

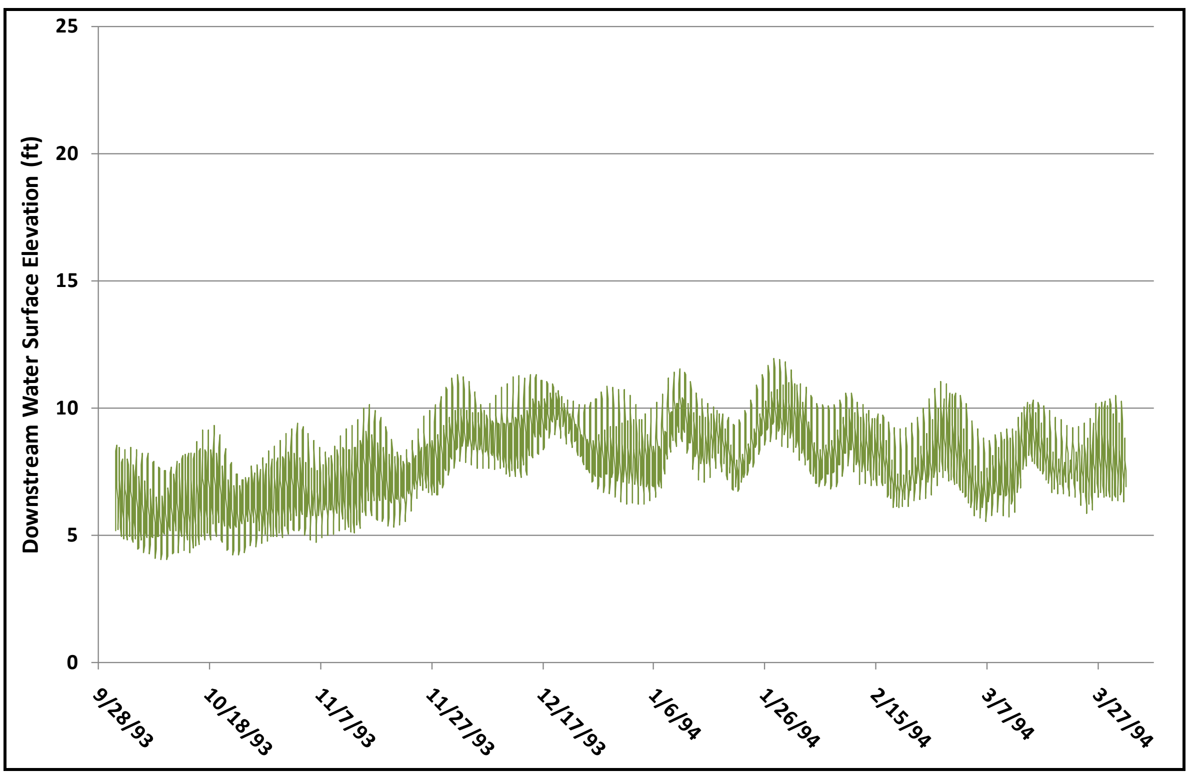


Figure 6. Boundary Condition Columbia River Water Surface Elevation - WY 1994

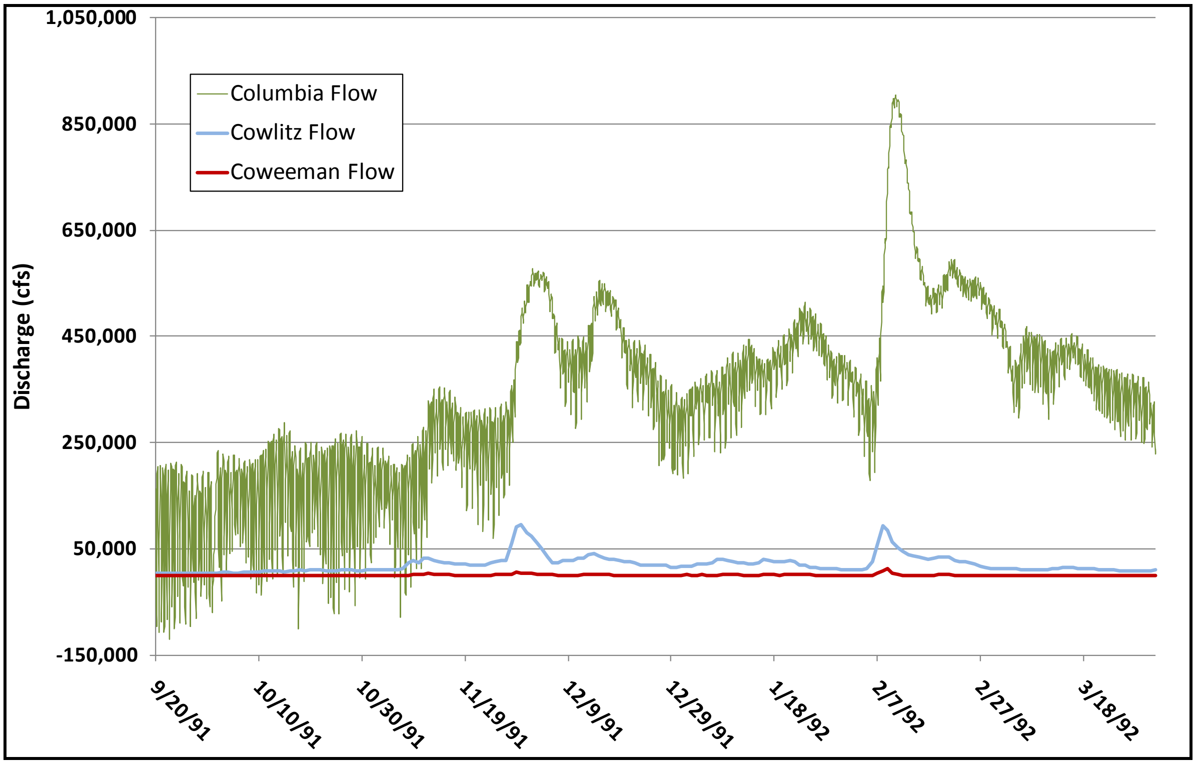


Figure 7. Boundary Condition Flows from Columbia, Cowlitz, and Coweeman Rivers - WY 1992

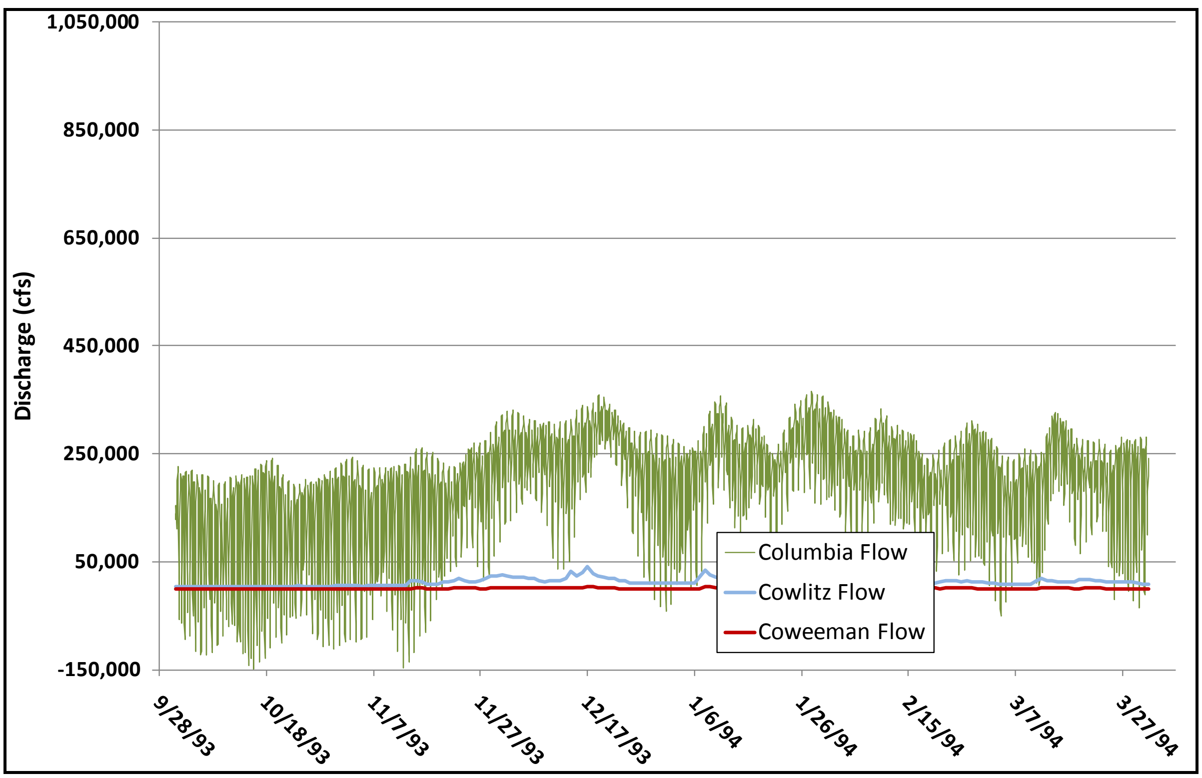


Figure 8. Boundary Condition Flows from Columbia, Cowlitz, and Coweeman Rivers - WY 1994

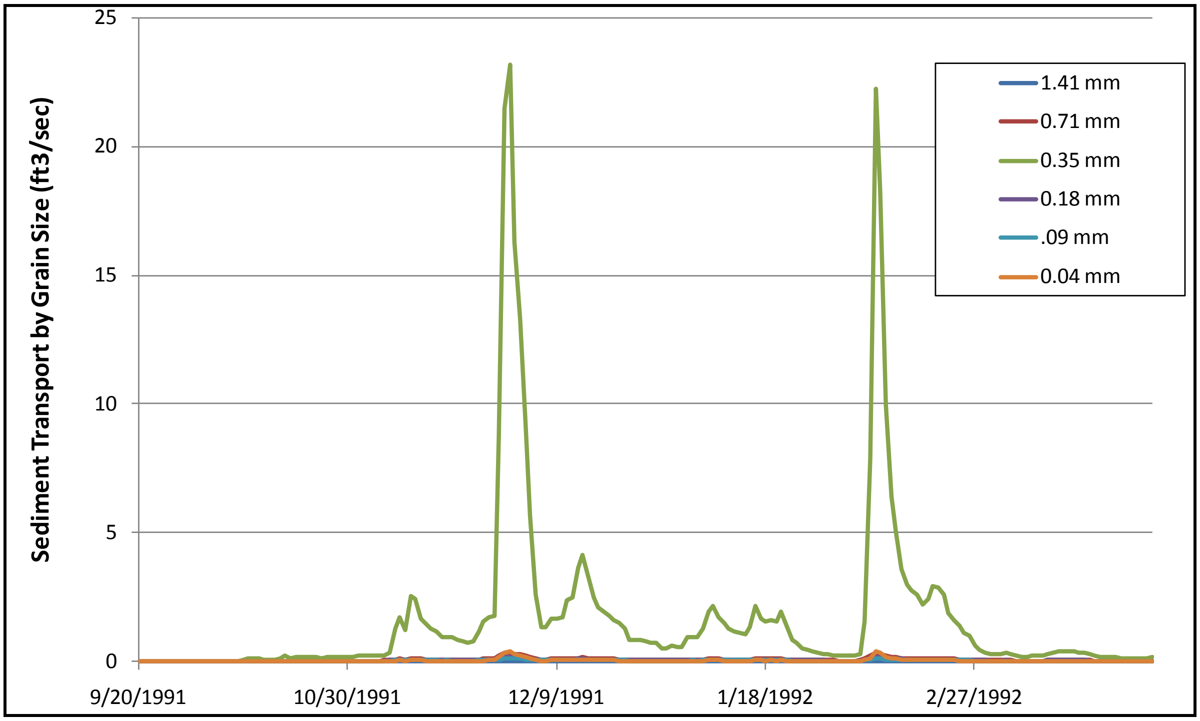


Figure 9. Sediment inflow by grain size to the Cowlitz River - WY 1992

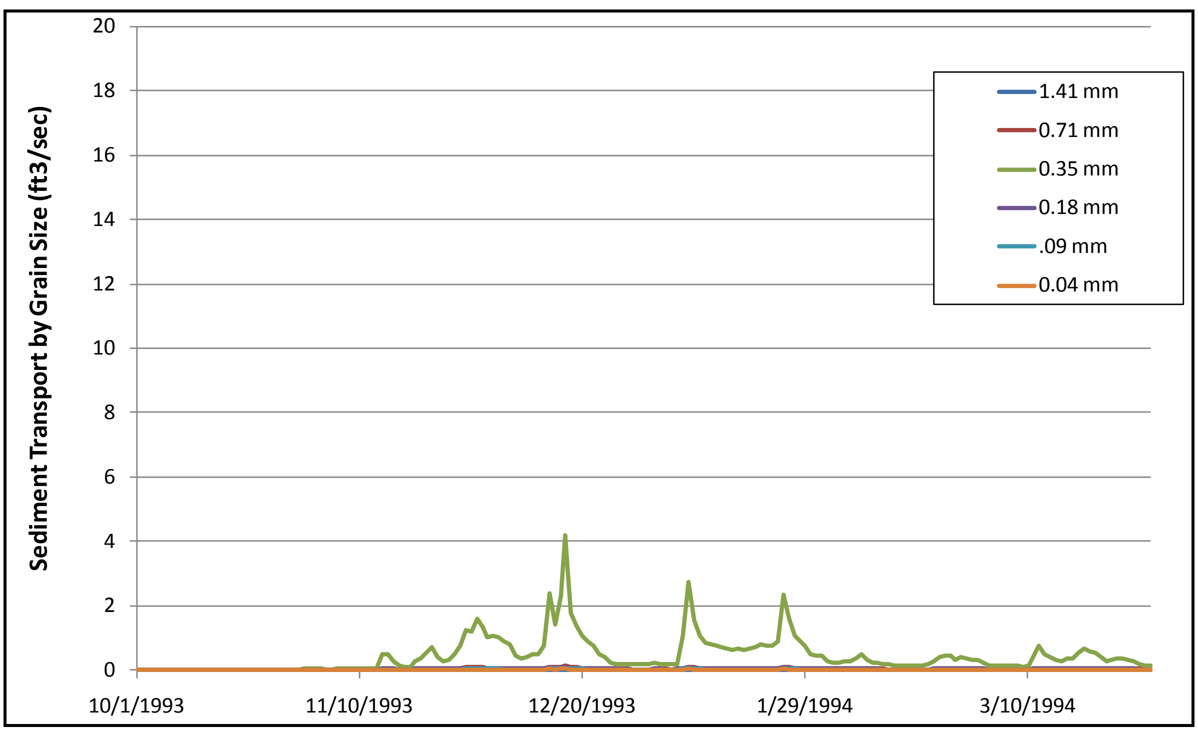


Figure 10. Sediment inflow by grain size to the Cowlitz River - WY 1994

RESULTS

High Flow Water Year - 1992

Sediment transport effectiveness of the dike system was evaluated for a high Cowlitz River flow year by comparing deposition volumes within four reaches of the Lower Cowlitz River for five observation periods within the 1992 water year hydrograph. **Figure 11** shows the spatial reach breakdown of Reaches 1 through 4 (upstream to downstream). These areas were selected as areas of interest since Reach 1 is the initial dike field at the upstream end of the model, Reach 2 includes a large radius left bend with long dikes, Reach 3 is a long right bend with small dikes, and Reach 4 is the downstream most area near the mouth of the Cowlitz, Carol's Channel, and the confluence with the Columbia River.

The high flow period of interest included flows above base level for water year 1992 and was divided into 5 key observation periods. The first period (Observation Period 1) is characterized by low flow leading to a large peak flow period (Observation Period 2), a medium high flow period (Observation Period 3), another large peak (Observation Period 4), and the receding limb of the hydrograph (Observation Period 5). These temporal divisions can be seen in **Figure 12**.

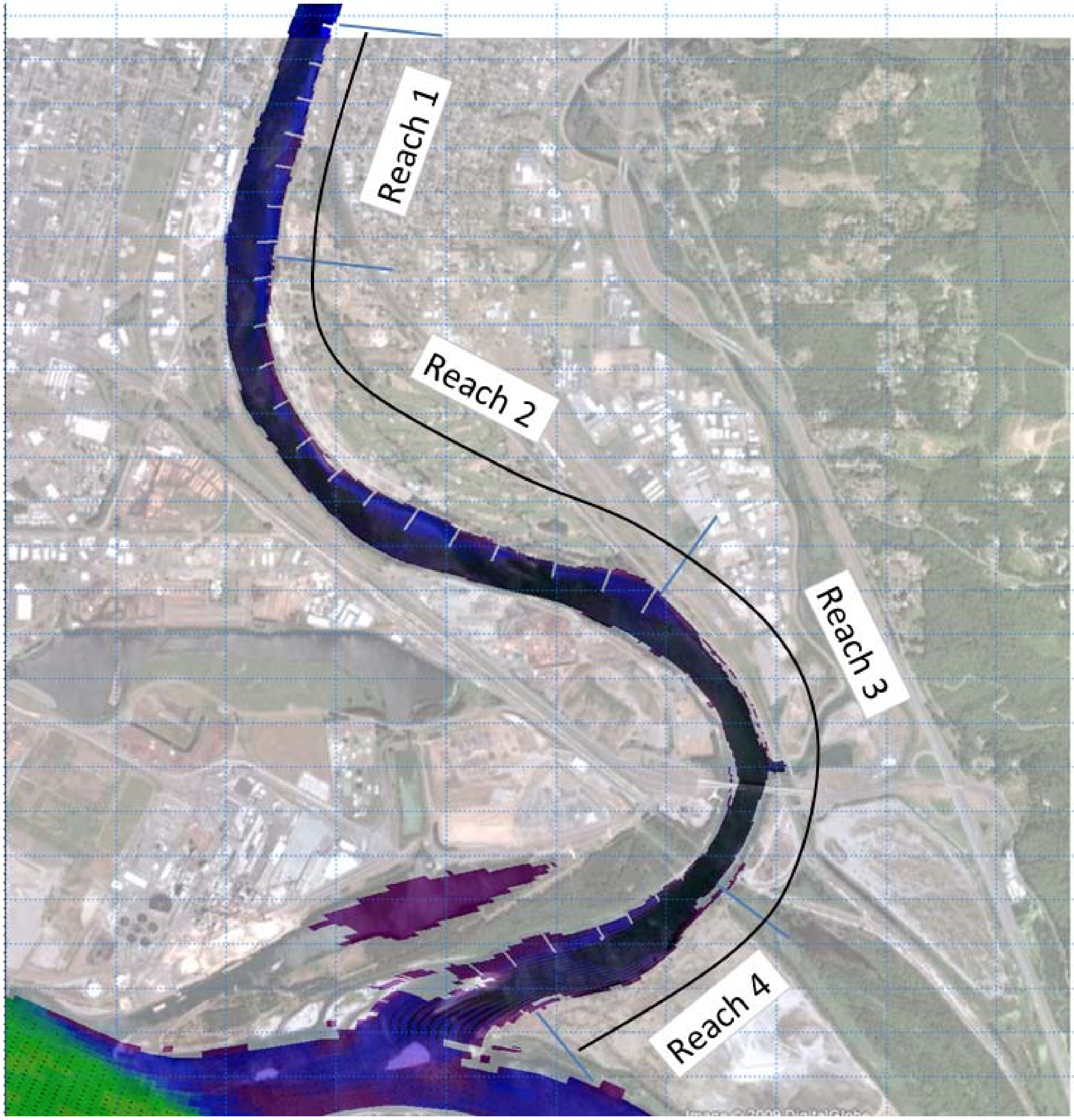


Figure 11. Lower Cowlitz Dike Study Reach Delineation

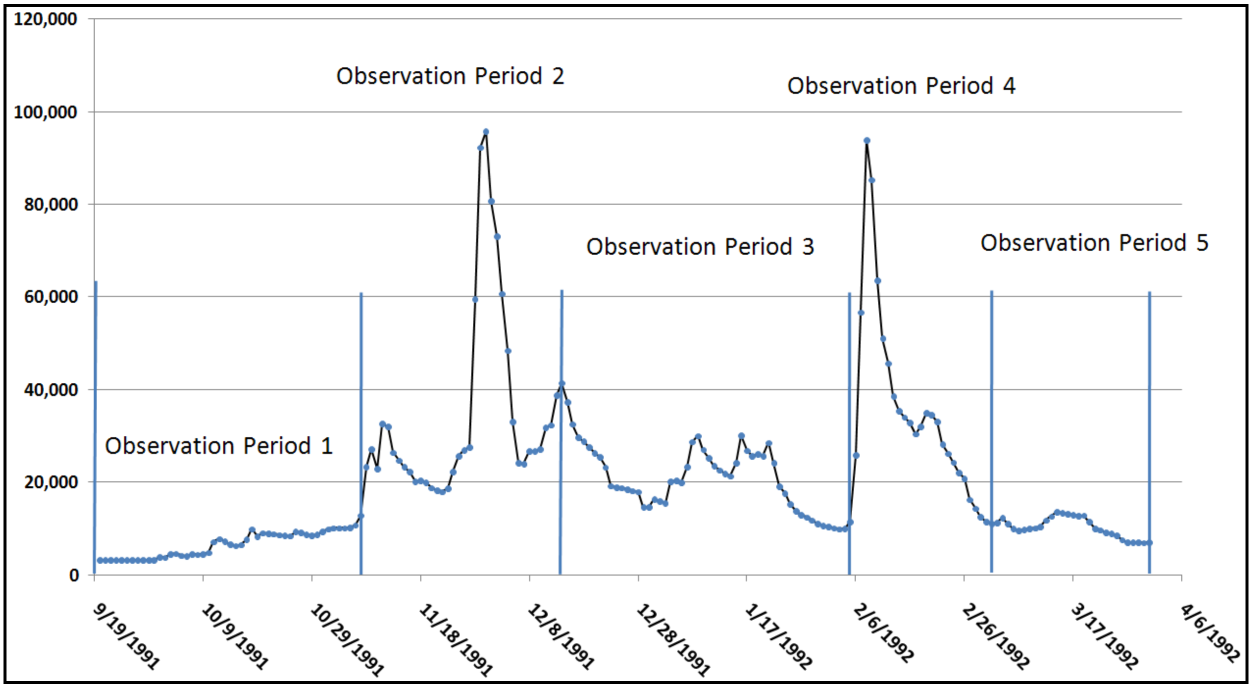


Figure 12. Observation Periods within the WY 1992 Cowlitz River Flow Hydrograph

Sediment volumes were calculated per reach by subtracting individual cell bed elevations at the end of an observation period from the initial bed elevations at the beginning of that observation period and multiplying by individual cell areas. The incremental changes were combined for the reaches within the active channel, or the area where dredging would be anticipated. **Figure 13** shows the area of the channel that was assumed to be the active channel, all volume comparisons are based on bed change (deposition and scour) within this zone.

High Flow Water Year Existing Case - Baseline Model

The baseline without dikes model of the Lower Cowlitz is capable of scouring for the most part over the October, 1991 to March, 1992 study period. As **Table 1** shows, aside from Reach 1 which may deposit 430,750 CY of sediment over the study period, Reaches 2, 3, and 4 are expected to scour. Observation Period 1 is slightly depositional for Reach 1 (30 CY), but scours in Reaches 2, and 3 (1,900 and 72,300 CY respectively), and deposits in Reach 4 (38,000 CY). A total of 36,100 CY is expected to scour in Observation Period 1.

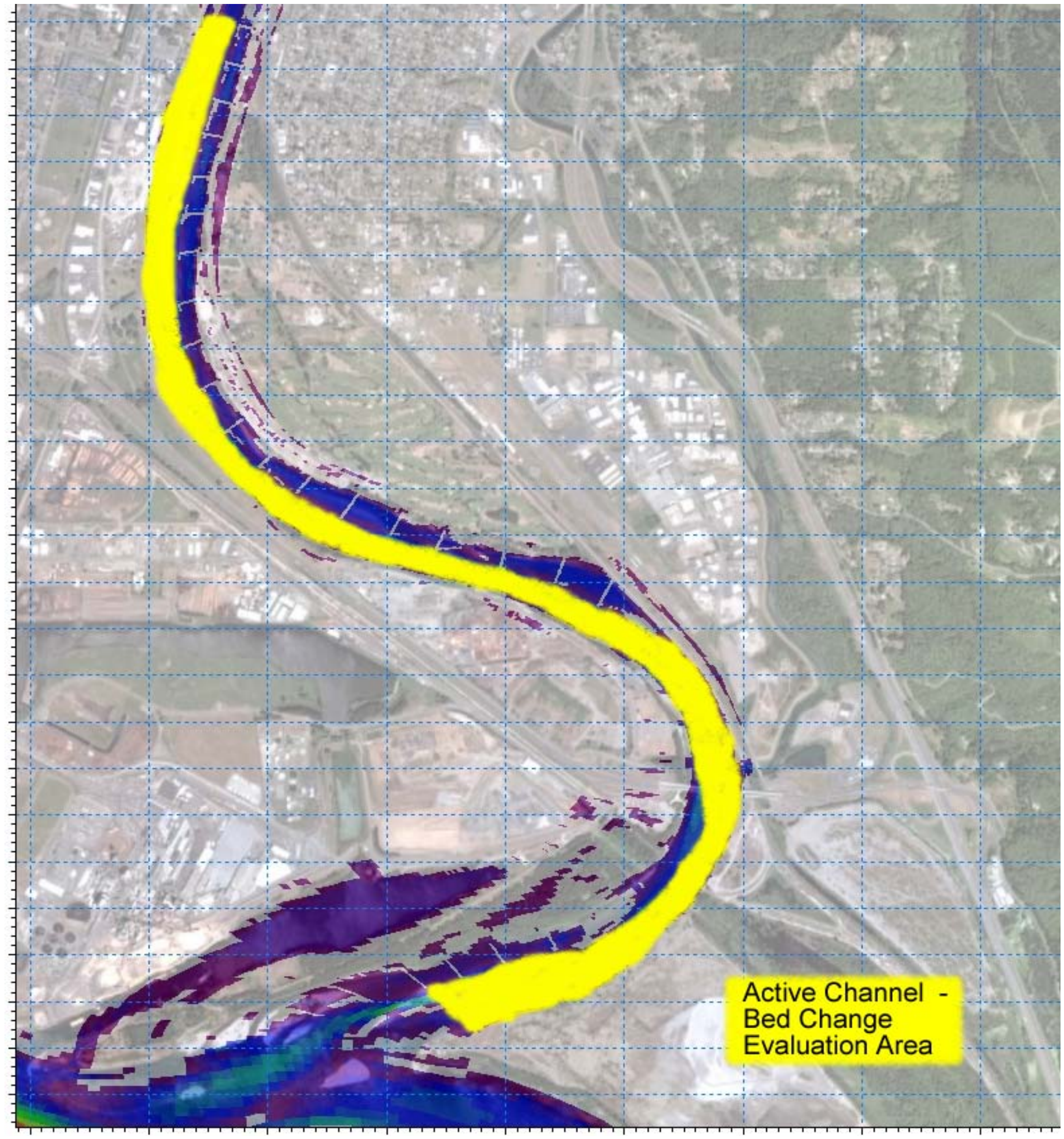


Figure 13. Lower Cowlitz River Active Channel Evaluation Area

Observation Period 2 (the first peak discharge period, see **Figure 12**), shows that the high flows (up to 96,000 cfs) result in a large amount of deposition in Reach 1 (224,200 CY), and large scours in Reaches 2, 3, and 4 (241,600 CY; 418,300 CY; and 249,900 CY respectively). Observation Period 2 results in over 685,500 CY of scour for the combined reaches of the Lower Cowlitz River.

During the third Observation Period primarily deposition occurs after the large scour from the peak flow during Observation Period 2. The total deposition for all reaches is over 120,300 cubic

yards. Reach 1 can deposit 136,400 CY, Reaches 2, 3, and 4 are relatively inactive (4,500 CY of scour for Reach 2; 3,200 CY deposition for Reach 3; and 14,700 CY scour for Reach 4). Overall , Observation Period 3 results in 120,400 CY of deposition.

Baseline Case without Pile Dikes (***scour is negative**)

Observation Period	Reach 1	Reach 2	Reach 3	Reach 4	
1	33	(1,895)	(72,321)	38,060	(36,125)
2	224,200	(241,586)	(418,334)	(249,868)	(685,587)
3	136,428	(4,528)	3,238	(14,741)	120,397
4	70,371	123,485	(102,212)	(39,422)	52,222
5	2,726	13,526	219	(91)	16,379
	433,758	(110,998)	(589,410)	(266,062)	(532,714)

Table 1. Scour and Deposition by Reach and Observation Period - Baseline Case (cubic yards)

Observation Periods 4 and 5 are both depositional for the existing condition. Total deposition for Observation Period 4 is 52,200 CY, and for Observation Period 5 is 16,400 CY. In the second peak discharge portion of the hydrograph (Observation Period 4), the first two Reaches are depositional (70,400 CY for Reach 1 and 123,500 CY for Reach 2). Reaches 3 and 4 are scouring (102,200 and 39,400 CY respectively). Observation Period 5 is depositional in all reaches except Reach 4 (2,700 CY for Reach 1; 13,500 CY for Reach 2; and 200 CY for Reach 3). Reach 4 is scours slightly (90 CY).

Due to the large amount of scour that is expected during the first peak flow period (Observation Period 2), the system is expected to be efficient at transporting sediment throughout the study period. The existing condition model shows the ability to transport 532,700 CY more sediment than is expected to deposit during the study period.

High Flow Year Proposed Case - Dike Model

The dike model of the Lower Cowlitz shows the same trends for scour and deposition throughout the Observation Periods, except for a notable difference during Observation Period 4. The dike model encourages a large overall scouring trend during the second peak (Observation Period 4), whereas the baseline case showed deposition during this period.

Table 2 shows Observation Periods 1 and 2 generally scour in the dike model, although there is noticeably more scour during Observation Period 2 (1,460,000 CY as opposed to the 685,600 CY in the baseline case). As previously, Reaches 1 and 4 are depositional during Observation Period 1 (150CY; and 55,000 CY respectively), while Reaches 2 and 3 are shown to scour (33,800 and 61,500 CY respectively). Even the total scour is similar quantitatively to the baseline case: 40,100 CY with dikes, and 36,100 CY without for Observation Period 1. Since the same trend is apparent during the last Observation Period, Observation Period 5, this could lead to the conclusion that dikes may not be very effective at lower flow levels within this study period (less than about 10,000 cfs within the Cowlitz River).

Pier Dike Case Sediment Change per zone (scour is negative)

Obervation Period	<-----------Reach-----------> 1	2	3	4	
1	149	(33,775)	(61,468)	54,976	(40,118)
2	67,259	(957,300)	(321,785)	(248,416)	(1,460,242)
3	132,378	51,778	(30,808)	(22,711)	130,638
4	4,399	68,851	(61,323)	(219,753)	(207,826)
5	12,853	5,412	(11)	391	18,645
	217,037	(865,033)	(475,394)	(435,513)	(1,558,903)

Table 2. Scour and Deposition by Reach and Observation Period - Dike Case (cubic yards)

Observation Period 2 is the longest period of scour, and over twice as much scour is expected with the dikes (1,460,200 CY versus 685,600 CY without the dikes). The reach trends are the same as the baseline case, Reach 1 is depositional (although less depositional than the baseline

case); 67,000 CY of sediment is expected to deposit in Reach 1. Reaches 2, 3, and 4 are scouring (957,300 CY; 321,800 CY; and 248,400 CY respectively).

Reaches 3 and 4 are expected to scour, but the overall study area is depositional during the medium flow Observation Period 3 (130,600 CY is expected to deposit in Reaches 1-4). The depositional volume is similar between the baseline case (120,400 CY) and the dike case (130,600 CY) during this period.

The most significant difference between the dike model and the baseline model is the ability of the dikes to cause a large amount of scour during Observation Period 4 (207,800 CY scour versus 52,200 cy of deposition without the dikes). Reach 1 is slightly depositional (4,400 CY), and Reach 2 is expected to deposit 68,900 CY. As with the baseline case, both Reaches 3 and 4 are expected to scour during Observation Period 4 (61,300 and 219,800 CY respectively).

Observation Period 5 is very similar to the baseline case. All reaches are slightly depositional except Reach 3 which is almost inactive (11 CY scour). The dike model predicts a total deposition of 18,600 CY during Observation Period 5.

Throughout the study period, the dike model is almost three times as effective at moving sediment than the baseline model (1,558,900 CY scour with dikes; 532,700 CY scour without dikes). This trend is expected to hold true even if subsequent model calibration results in both models being depositional.

Comparison - Dike Effectiveness

The dikes as initially modeled may extend up to 300 feet from the river bank into the channel through the upper reaches (Reaches 1 and 2). At high flow levels the large dikes in these reaches effectively concentrate flows and enable increased sediment transport. The downstream dikes are not as long, but with the exception of Reach 3, which has very short dikes, they are still effective in their ability to increase sediment transport through the downstream reaches. **Table 3** shows a direct comparison between the results of the baseline and dike models.

Pier Effectiveness
(increased sediment transport efficiency is positive)

Obervation Period	Reach 1	Reach 2	Reach 3	Reach 4	
1	(116)	31,879	(10,853)	(16,916)	3,994
2	156,942	715,714	(96,549)	(1,452)	774,655
3	4,049	(56,306)	34,046	7,970	(10,241)
4	65,973	54,634	(40,889)	180,331	260,048
5	(10,127)	8,114	229	(483)	(2,266)
	216,720	754,035	(114,016)	169,451	1,026,190

Table 3. Effectiveness of Dikes to Encourage Sediment Mobility (cubic yards)

During Observation Periods 1, 2, and 4 the dike model shows more overall scour potential with the dikes than the baseline model (4,000 CY; 774,700 CY; and 260,000 CY respectively). The dike model shows slightly more deposition during Observation Periods 3 and 5, although both of these values (10,200 CY for Observation Period 3 and 2,300 CY for Observation Period 5) are relatively small compared with the large scour values during the peak flow periods Observation Periods 2 and 4).

Reaches 1, 2, and 4 show increased sediment transport efficiency in the dike model. Each of these reaches shows significantly higher transport rates during the study period (216,700 CY for Reach 1, 754,000 CY for Reach 2, and 169,400 CY for Reach 4). Reach 3 (with the smaller dikes) shows 114,000 CY more deposition than the without dike model. Some of the scour from Reaches 1 and 2 may settle in Reach 3, and refinement of the dike configuration may enhance sediment transport performance in that region.

Typical Water Year - 1994

To quantify sediment transport effectiveness of the dike system for a common Cowlitz River flow year, deposition volumes were compared within four reaches of the Lower Cowlitz River for five observation periods during the more typical 1994 water year hydrograph.

The 1994 water year study period included flows above base level for water year 1994 and was divided into 5 key observation periods. The first period (Observation Period 1) is characterized by low flow leading to a large peak flow period (Observation Period 2), a medium flow period (Observation Period 3), another set of peaks (Observation Period 4), and the medium flow period following them (Observation Period 5). These Observation Periods can be seen in **Figure 14**.

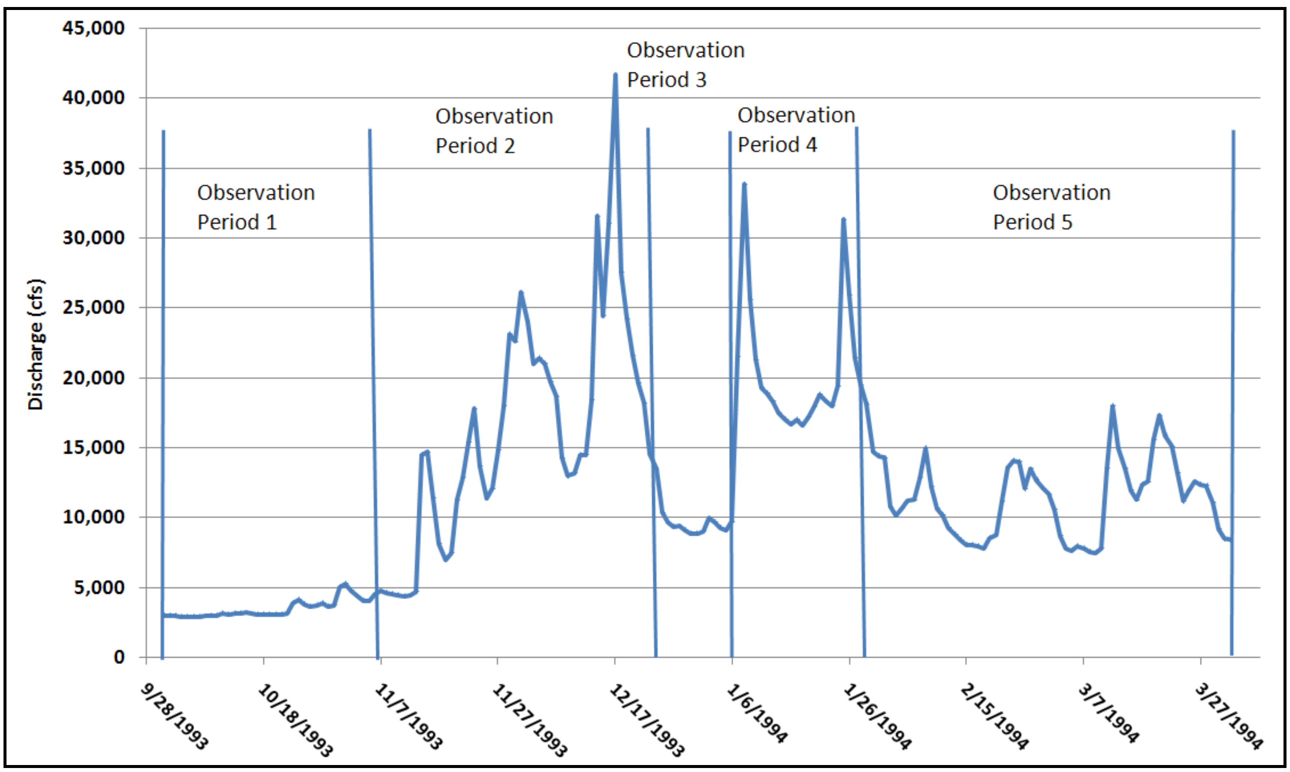


Figure 14. Observation Periods within the WY 1994 Cowlitz River Flow Hydrograph

Existing Case - Baseline Model

The baseline without dikes model of the Lower Cowlitz is capable of scouring overall for the October, 1993 to April, 1994 study period. As **Table 4** shows, Reaches 1 and 4 are depositional and Reaches 2, 3 are expected to scour. Observation Period 1 is stable in Reach 1, but scours in Reaches 2, and 3 (40 CY and 16,600 CY respectively), and deposits in Reach 4 (2,200 CY). A total of 14,500 CY is expected to scour throughout the study area in Observation Period 1.

Baseline Case without Pier Dikes (scour is negative)

Obervation Period	Reach 1	Reach 2	Reach 3	Reach 4	
1	0	(40)	(16,628)	2,199	(14,470)
2	11,586	(28,905)	(295,439)	173,096	(139,662)
3	4,435	(22,911)	(10,920)	(43,850)	(73,247)
4	9,027	(20,299)	(2,432)	(25,469)	(39,174)
5	46,340	(13,741)	2,681	(29,682)	5,598
	71,388	(85,896)	(322,739)	76,293	(260,954)

Table 4. Scour and Deposition by Reach and Observation Period - Baseline Case (cubic yards)

Observation Period 2 (the first peak discharge period, see **Figure 14**), shows that the high flows (up to 42,000 cfs) result in deposition in Reach 1 (11,600 CY), and scour in Reaches 2 and 3 (28,900 CY, and 295,400 CY respectively). Reach 4 is depositional (173,100 CY). Observation Period 2 results in over 139,600 CY of scour for the combined reaches of the Lower Cowlitz River.

During the third Observation Period of medium flow, the scour trend continues. The total scour for all reaches is over 73,200 cubic yards. Reach 1 can deposit 4,400 CY, and Reaches 2, 3, and 4 are scouring (22,900 CY of scour for Reach 2; 10,900 CY scour for Reach 3; and 43,800 CY scour for Reach 4).

Observation Period 4 continues scouring and Observation Period 5 is slightly depositional for the existing condition. Total scour for Observation Period 4 is 39,200 CY, and for Observation Period 5 deposition is 5,600 CY. In the second peak discharge portion of the hydrograph (Observation

Period 4), the first reach is depositional (9,000 CY). Reaches 2, 3, and 4 are scouring (20,300 for Reach 2; 2,400 for Reach 3; and 25,500 CY for Reach 4). Observation Period 5 is almost stable with a total deposition of 5,600 CY (46,300 CY deposition in Reach 1; 13,700 CY scour in Reach 2; 2,700 CY deposition in Reach 3; and 29,700 CY scour in Reach 4).

Since overall scour is predicted in each Observation Period except 5, the Baseline Case system is expected to be efficient at transporting sediment throughout the 1994 water year study period. The existing condition model shows the ability to transport 260,950 CY more sediment than is expected to deposit during this study period.

Proposed Case - Dike Model

The dike model of the Lower Cowlitz scours throughout each Observation Period, and is noticeably more efficient at moving sediment during both peak periods (Observation Periods 2 and 4). **Table 5** shows Observation Periods 1 and 2 generally scour in the dike model, although there is a considerable amount of deposition in Reach 4 during Observation Period 2. As previously, Reach 1 is stable and Reach 4 is slightly depositional during Observation Period 1 (5,100 CY of deposition in Reach 4), while Reaches 2 and 3 are shown to scour (8,000 and 15,400 CY respectively). The total scour is similar quantitatively to the baseline case: 18,300 CY with dikes compared to 14,500 CY without for Observation Period 1.

Pier Dike Case (scour is negative)

Obervation Period	<-----------Reach-----------> 1	2	3	4	
1	0	(8,008)	(15,359)	5,112	(18,255)
2	9,491	(106,161)	(262,407)	130,141	(228,936)
3	110	(9,053)	4,995	(3,556)	(7,504)
4	12,577	(77,885)	(16,121)	(69,997)	(151,426)
5	57,394	(45,044)	11,044	(32,643)	(9,249)
	79,573	(246,152)	(277,848)	29,057	(415,370)

Table 5. Scour and Deposition by Reach and Observation Period - Dike Case (cubic yards)

Observation Period 2 is the longest period of scour, and almost twice as much scour is expected with the dikes (228,900 CY versus 139,700 CY without the dikes). The reach trends are the same as the baseline case, Reaches 1 and 4 are depositional (9,500 CY of sediment is expected to deposit in Reach 1, and 130,100 CY in Reach 4). Reaches 2 and 3 are scouring (106,200 CY; and 262,400 CY respectively).

The study area is fairly stable in Observation Period 3. Reaches 2 and 4 are expected to scour (9,100 CY and 3,600 CY respectively) Reaches 1 and 3 are depositional during the medium flow Observation Period 3 (100 CY is expected to deposit in Reach 1, and 5,000 CY in Reach 3). The study area scours overall (7,500 CY) during this period.

As was noted in the 1994 water year model comparison, the dikes to cause a large amount of scour during peak flows in Observation Periods 2 and 4 (228,900 CY scour versus 139,700 CY of scour without the dikes in Observation Period 2, and 151,400 CY of scour versus 39,200 CY scour without the dikes in Observation Period 4). Reach 1 is slightly depositional (12,600 CY), but Reaches 2, 3, and 4 are expected to scour (77,900 CY; 16,100 CY; and 70,000 CY respectively) during Observation Period 4.

Observation Period 5 is very similar to the baseline case. Reaches 1 and 3 are depositional (57,400 CY in Reach 1 and 11,000 CY in Reach 3) and Reaches 2 and 4 are scouring (45,000 CY in Reach 2 and 32,600 in Reach 4). The dike model predicts a total scour of 9,200 CY during Observation Period 5.

Throughout the 1994 water year study period, the dike model is over one and a half times as effective at moving sediment than the baseline model (415,400 CY scour with dikes; 261,000 CY scour without dikes).

Comparison - Dike Effectiveness

A direct comparison between the results of the baseline and dike models is shown in **Table 6**.

Dike Effectiveness
(increased sediment transport effiency is positive)

Obervation Period	<-----------Reach----------->				
	1	2	3	4	
1	(0)	7,968	(1,269)	(2,913)	3,785
2	2,095	77,257	(33,032)	42,955	89,274
3	4,325	(13,858)	(15,915)	(40,295)	(65,743)
4	(3,551)	57,587	13,689	44,528	112,253
5	(11,054)	31,303	(8,363)	2,961	14,847
	(8,185)	160,256	(44,891)	47,236	154,416

Table 6. Effectiveness of Dikes to Encourage Sediment Mobility (cubic yards)

During Observation Periods 1, 2, 4, and 5 the dike model shows more sediment transport than the baseline model (3,800 CY; 89,300 CY; 112,300 CY; and 14,800 CY more respectively). The dike model shows less transport capacity during the third Observation Period (65,700 CY) between the two peak flow periods (Observation Periods 2 and 4).

Reaches 2, and 4 show considerable increased sediment transport effectiveness in the dike model. These reaches show significantly higher transport rates during the study period (160,300 CY for Reach 1, and 47,200 CY for Reach 4). Reaches 1 and 3 show 8,100 CY and 44,900 CY less sediment transport than the baseline model. Reach 1 is depositional in both the baseline and with piers models. Refinement of the dike configuration may enhance sediment transport performance in Reach 1 to help minimize deposition in these areas. Since Reach 4 is depositional even with the dikes (although dikes decrease the deposition from 76,300 CY to 29,000 CY, further refinement of the dikes in Reach 4 may help to decrease deposition in this area as well.

Summary

The preliminary model results, which compare the baseline and dike models for high flow and typical flow years on the Cowlitz River, show that for the both high flow and typical flow study periods dikes are predicted to notably increase the sediment transport efficiency of the Lower Cowlitz River. These results indicate that at low Cowlitz River discharge levels the dike model

moves sediment at a similar rate to the baseline model (both high flow and typical flow Observation Periods 1 have similar dike and baseline sediment transport capacity with and without dikes), but at medium to high Cowlitz River flows (including high Cowlitz River flow periods during typical years) the dikes could decrease dredging operations by facilitating almost twice to three times as much sediment transport through the system down to the Cowlitz/Columbia River confluence.

Recommendations

The initial Lower Cowlitz dike simulation models were part of a preliminary effort intended to show the effectiveness of dikes in moving sediment through the Lower Cowlitz River and to demonstrate that it could be evaluated with a 2-dimensional hydrodynamic sediment model. Moving forward, several additional levels of refinement will lead to a more detailed understanding of dike performance:

1) A key aspect of dike design is to balance the need for increased sediment transport capacity at the low to intermediate stages with the requirement that flood stages not be significantly increased at higher flows. Refinement of the dike field is necessary to result in lower stages at lower flows, with minimal changes in stage at the higher flows.

2) The Cowlitz River flow hydrology used for this study was for two discreet years - one high flow year of above baseline flow and one typical flow year above baseline flow. Several years modeled in sequence may help to understand performance of the dike system over time. Studying the cumulative results of several years would yield conclusions on the benefits of dikes on long term channel maintenance.

3) Results from this study can be used to fine tune the initial dike field layout. Fewer and smaller dikes can be simulated through Reaches 1 and 2. More effective dike placement could be developed through Reach 3. The basic models developed for this study can be adapted to simulate any number of potential dike scenarios.

4) Specific dredge channels can be modeled with this approach. A series of runs with dredged channel bed geometry in the baseline case could be evaluated and compared with the dike alternatives to understand how quickly dredged channels would require re-dredging with and without dikes.